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This report includes ten papers, on various model - related topics, presented at the semi-annual joint U.S. - Canadian Stormwater Management Model (SWMM) Users Group Meeting, held 13-14 November 1978 in Annapolis, Maryland.

Topics covered include a description of the new SWMM Storage/Treatment program, several papers on stormwater retention, a methodology for evaluating agricultural BMP's, two papers on model applications in facility planning, and other papers on model applications.

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STORMWATER MANAGEMENT MODEL (SWMM) USERS GROUP MEETING
13-14 NOVEMBER 1978

Project Officer

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OFFICE OF AIR, LAND, AND WATER USE
OFFICE OF RESEARCH AND DEVELOPMENT
U.S. ENVIRONMENTAL PROTECTION AGENCY
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FOREWORD

A major function of the Research and Development program of the Environmental Protection Agency is to effectively and expeditiously transfer, to the user community, technology developed by those programs. A corollary function is to provide for the continuing exchange of information and ideas between EPA and users, and between the users themselves. The Stormwater Management Model (SWMM) users group, sponsored jointly with Environment Canada/Ontario Ministry of the Environment, was established to provide such a forum.

This report, a compendium of papers presented at the last Users Group meeting, is published in the interest of disseminating to a wide audience the work of Group members.

Courtney Riordan
Acting Deputy Assistant Administrator
Office of Air, Land and Water Use

ABSTRACT

This report includes ten papers, on various model - related topics, presented at the semi-annual joint U.S. - Canadian Stormwater Management Model (SWMM) Users Group Meeting, held November 13-14, 1978 in Annapolis, Maryland.

Topics covered include a description of the new SWMM Storage/Treatment program, several papers on stormwater retention, a methodology for evaluating agricultural BMP's, two papers on model applications in facility planning, and other papers on model applications.

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REVISED SWMM STORAGE/TREATMENT BLOCK

by

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University of Florida
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Presented at the
Storm Water Management Model Users
Group Meeting

Annapolis, Maryland
November 13-14, 1978

PREFACE

The following paper is Section 5 of a forthcoming documentation report of SWMM procedures for the Environmental Protection Agency. The research has been supported by EPA Grants R-802411 and R-805664.

SECTION 5

STORAGE/TREATMENT SIMULATION

OBJECTIVES

The primary objectives of the Storage/Treatment Block are to:

1. provide the capability of modeling a larger number of processes in both the single event and continuous modes;
2. simulate the quality improvement provided by each process;
3. simulate the handling of sludges; and
4. provide estimates of capital, operation and maintenance costs.

Although the objectives of the Storage/Treatment Block have not changed appreciably from earlier versions (1), the model has been virtually rewritten. The earlier versions were too limited in use and scope. This version is much more flexible in terms of the control units available, pollutant routing and cost estimating. However, the user is advised that increased flexibility implies increased user input and knowledge of the processes to be modeled. In other words, the model does not provide several dozen specialized designs, but provides the tools necessary to simulate the desired processes. Naturally, flexibility precludes ultrasophistication.

Several precautions should be noted before setting up the S/T Block.

1. The need for local waste characterization data cannot be overemphazised. Treatment unit performance is a function of the nature of the waste.
2. Lab or pilot plant performance data should be used whenever possible to derive performance functions.
3. Dry-weather treatment performance functions should be applied cautiously to wet-weather units.

PROGRAM DEVELOPMENT AND OVERVIEW

Development

Past versions of the Storage/Treatment Block simulated various processes on the basis of limited empirical data and operating experience.

Often the data were localized and/or specialized. Thus, they were of questionable applicability to a wide variety of situations. Additionally, the model did not account for the physical characteristics of the incoming waste stream or the handling of sludges.

To improve the storage/treatment modeling capabilities of SWMM the following considerations were instrumental in creating a new model.

1. There should be a high degree of flexibility in the simulation of individual units and the interaction among units.
2. In addition to simulating the mass of pollutants, it is important to account for the physical characteristics (i.e., particle size and specific gravity distribution) of each pollutant.
3. Sludge handling is an important part of any wastewater treatment scheme and should be simulated.
4. All costing routines should be as flexible as the performance algorithms.
5. The model should be capable of modeling wet- and dry-weather facilities.

Overview

The present Storage/Treatment Block is approximately 1500 Fortran statements in length and consists of eight subroutines. The routing of flow and pollutants through the entire block is controlled by subroutine STRT which is called from the Executive Block. STRT also provides the main driving loop for the model and generally acts as the central coordinating subroutine. Subroutine STRDAT is called in STRT and is responsible for reading the input data provided by the user. Subroutine CONTRL is called each time-step from the main driving loop in STRT. CONTRL directs flow and pollutants from one unit to another as prescribed by the desired scheme and coordinates the majority of the printed output. Subroutine UNIT is called from CONTRL for each unit modeled and is the heart of the Storage/Treatment Block. It contains the necessary flexibility and capability to model most storage/treatment processes (units). Subroutine EQUATE is used by UNIT to provide several forms of pollutant removal equations. Subroutine INTERP is employed by UNIT for linear interpolation. Subroutine PLUGS is used by UNIT to model perfect plug flow through a detention unit. Subroutine STCOST is called from STRT to determine capital, and operation and maintenance costs.

The model has become user-intensive rather than program-intensive. The user is responsible for providing the program with the desired storage/treatment scheme and operating characteristics of each unit (along with other information). However, input guidelines are provided in the User's Manual for several types of units. Again, the strength of this approach is to maximize flexibility and applicability to local conditions and design criteria.

SIMULATION TECHNIQUES

Introduction

Flow and pollutants are routed through one or more storage/treatment units by several techniques. The units may be arranged in any fashion, restricted only by the requirements that inflow to the plant enters at only one unit and that the products (treated outflow, residuals, and bypass flow) from each unit not be directed to more than three units. The flows into, through and out of a unit are shown in Figure 5-1. Treatment and sludge handling units are modeled by the same subroutine (UNIT). Additionally, both wet- and dry-weather facilities may be simulated by the proper selection of unit arrangement and characteristics. Units may be modeled as having a detention capability or instantaneous throughflow. Pollutants or sludges may be represented as a simple mass or further characterized by a particle size distribution. A unit may remove pollutants (or concentrate sludges) as a function of particle size and specific gravity, detention time, incoming concentration, the removal rate of another pollutant, or a constant percentage. The S/T Block can receive the flow and any three pollutants from any one outlet in any other block of SWMM. Also, flows and pollutants may be provided by the user and fed directly to the S/T Block. If both sources are present they are combined and treated as one input. For example, the user may enter directly dry-weather flows and enter wet-weather flows from the Runoff Block. All flows and pollutants in the S/T Block are assumed to be averages over a time step. This includes the input data and internal calculations.

The following sections describe the techniques available for flow and pollutant routing which allow the user to model several types of storage/treatment units.

Flow Routing

Detention vs. Instantaneous Throughflow --

A unit may be modeled to handle flow in one of two ways; as a detention basin (reservoir) or a unit instantaneously passing all flow. The idea of a detention basin is not limited to storage basins and sedimentation tanks but also includes such processes as dissolved air flotation, activated sludge, and chlorination. Processes that may be modeled as having instantaneous through-flow include microscreens, fine screens and other forms of screening.

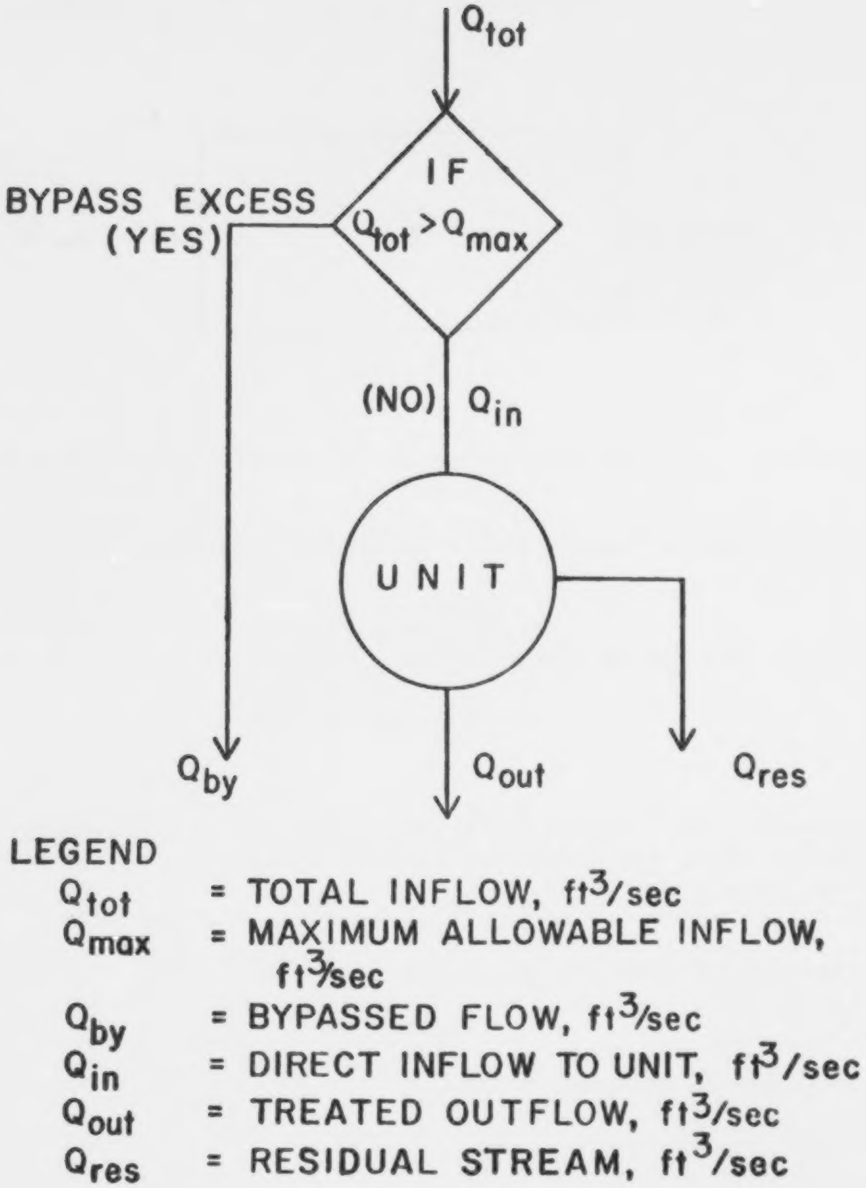
Q_{tot}
IF $Q_{tot} > Q_{max}$
BYPASS EXCESS (YES)
(NO) Q_{in}
UNIT
Q_{by}
Q_{out}
Q_{res}
LEGEND
Q_{tot} = TOTAL INFLOW, ft^3/sec
Q_{max} = MAXIMUM ALLOWABLE INFLOW, ft^3/sec
Q_{by} = BYPASSED FLOW, ft^3/sec
Q_{in} = DIRECT INFLOW TO UNIT, ft^3/sec
Q_{out} = TREATED OUTFLOW, ft^3/sec
Q_{res} = RESIDUAL STREAM, ft^3/sec

Figure 5-1. Flows Into, Through, and Out of a Storage/Treatment Unit.

Detention Basins --

The rate of change of storage in a detention basin is found by writing a mass balance equation for the system shown in Figure 5-2.

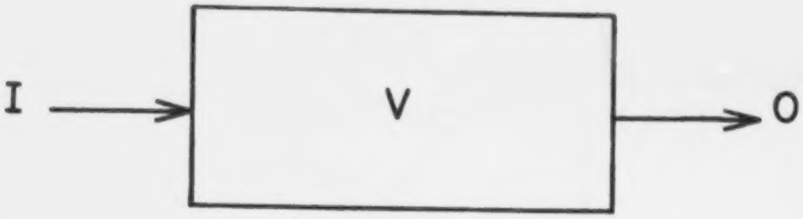

Figure 5-2. Time Varying Inflow and Outflow Rates for a Reservoir.

The rate of change of storage equals inflow minus outflow, or

$$\Delta V/\Delta t = \bar{I} - \bar{O} \tag{5-1}$$

where $\bar{I}$ = average inflow rate during Δt, ft^3/sec.

$\bar{O}$ = average outflow rate during Δt, ft^3/sec.
V = reservoir volume, ft^3, and
Δt = time step, seconds.

Let subscripts 1 and 2 denote the beginning and end of the time period, respectively. Then, the average inflow rate $\bar{I}$, is

$$\bar{I} = (I_1 + I_2)/2 \tag{5-2}$$

The average outflow rate, $\bar{O}$, is

$$\bar{O} = (O_1 + O_2)/2 \tag{5-3}$$

Also, the change in reservoir volume is

$$\Delta V = V_2 - V_1 \tag{5-4}$$

Substituting equations 5-2, 5-3, and 5-4 into equation 5-1 and multiplying through by Δt yields the desired expression for the change in volume, i.e.,

$$V_2 - V_1 = \frac{I_1 + I_2}{2} \Delta t - \frac{O_1 + O_2}{2} \Delta t . \tag{5-5}$$

For a given time step, I_1, I_2, O_1, and V_1 are known and O_2 and V_2 need to be determined. Grouping the unknowns on the left hand side of the equation and rearranging yields one of the two required equations:

$$(0.5)O_2 \,\Delta t + V_2 = 0.5(I_1 + I_2)\Delta t - [(0.5)\, O_1\Delta t - V_1] \qquad (5\text{-}6)$$

The second required equation is found by relating O_2 and V_2, each of which is a function of reservoir depth. The procedure is illustrated in the following example.

Table 5-1 presents geometric data for a hypothetical reservoir with a base elevation of 343.0 ft and a maximum pool elevation of 353.0 ft. The corresponding depths are shown in column 3. Surface area, as a function of depth, is presented in column 4. If the reservoir has an irregular geometry, the surface area is measured from a topographic map. The eleven depth-area data pairs shown in columns (3) and (4) of Table 5-1 are required input data. The model calculates the depth versus storage relationship by linear interpolation of the area between any two adjacent depth-area data pairs.

Recalling equation 5-6, the objective is to find

$$0.5\, O_2\, \Delta t = f(0.5\, O_2\Delta t + V_2) \qquad (5\text{-}7)$$

The data in Table 5-2 give O_2 and V_2 as functions of depth over the range of depths for which outflow occurs. In this case, outflow occurs only if the reservoir depth exceeds 8.0 ft. Thus, eleven data triplets may be selected which cover the range of interest. Then, as an option, the user inputs these eleven triplets of columns (3), Depth, (6), Outflow Volume (O2DT2), and (7), Outflow & Reservoir Volume (SATERM). During the simulation the model uses this relationship to estimate end-of-period storage and outflow.

Alternatively, the model will continue to generate parameters O2DT2 and SATERM from a specified weir, orifice or pumped outflow configuration. If a weir or orifice is at an elevation above the reservoir bottom (e.g., at elevation 351 ft in the example), the program will generate seven pairs of O2DT2 and SATERM at and above the weir/orifice elevation and four below.

In both cases, eleven depth-area pairs are also input. In the latter situation discussed above, these are used to calculate the depth-volume relationship for the reservoir. In the former situation in which the data triplets are input, the volume computation is not required. In both cases, the surface area is needed to estimate the volume of evaporation lost from the reservoir.

Table 5-1. Geometric Data For Hypothetical Reservoir

n (1)	Elevation h ft (2)	Depth y ft (3)	Surface Area A 1000 ft^2 (4)	Remarks (5)
1	343.	0	0	Base of reservoir
2	344.	1.	3.	
3	345.	2.	15.	
4	346.	3.	45.	
5	347.	4.	121.	
6	348.	5.	225.	
7	349.	6.	365.	
8	350.	7.	550.	
9	351.	8.	790.	Weir elevation
10	352.	9.	1080.	
11	353.	10.	1440.	Maximum pool

Column
(1) Counter
(2) Elevations from topographic map
(3) Depth = h - 343.
(4) Measured from topographic map or may be calculated (by user) if geometry is regular.

Table 5-2. Routing Data For Hypothetical Reservoir

n (1)	Elevation h ft (2)	Depth y ft (3)	Volume V_2 1000 ft^3 (4)	Discharge O_2 ft^3/sec (5)	O2DT2 $.5\ O_2\ \Delta t$ 1000 ft^3 (6)	SATERM $.5\ O_2\Delta t + V_2$ 1000 ft^3 (7)
1	351.0	8.0	1720.	0	0	1720.
2	351.2	8.2	1850.	10.	108.	1958.
3	351.4	8.4	2000.	20.	216.	2216.
4	351.6	8.6	2220.	35.	378.	2598.
5	351.8	8.8	2400.	50.	540.	2940.
6	352.0	9.0	2650.	65.	702.	3352.
7	352.2	9.2	2900.	80.	804.	3764.
8	352.4	9.4	3100.	105.	1134.	4234.
9	352.6	9.6	3400.	130.	1404.	4804.
10	352.8	9.8	3700.	165.	1782.	5482.
11	353.0	10.0	3900.	200.	2160.	6060.

Column
(1) Counter
(2) Elevation from topographic map
(3) Depth = h - 343.0
(4) Volume measured or calculated volume
(5) Measured data or calculated from discharge formulas
(6) Calculated using O_2 (col. 5), Δt = 21,600 sec
(7) Calculated using col. 4 and col. 6

The computational procedure is summarized as follows:

1. Known values of I_1, I_2, O_1, Δt, and V_1 are substituted into the right hand side of equation 5-6. The result is the first value of $0.5\ O_2\Delta t + V_2$.
2. Knowing $(0.5\ O_2\Delta t + V_2)$ the value of $0.5\ O_2\Delta t$ is obtained by interpolating between adjacent values of O2DT2 and SATERM.
3. Twice the value of $0.5\ O_2\Delta t$ is subtracted from V_2 to give the new $0.5\ O_1\Delta t - V_1$.
4. Add $0.5(I_1 + I_2)\ \Delta t$ to the new value of $0.5\ O_1\Delta t - V_1$ to get the new value of $0.5\ O_2\Delta t + V_2$.
5. Continue this process until all inflows have been routed.

To summarize the input alternatives, the earlier version of the storage model permitted the user to read in depth-area data and an outflow condition of a weir, orifice, or pumping. It could not handle the case of a natural reservoir with an irregular stage-discharge relationship. The updated model allows the user to input the required relationship between storage volume and outflow rate in the form of the eleven data triplets discussed previously. This approach permits the user to select the eleven data points which best approximate the desired functional relationship. This approach is felt to be preferable to adding more complexity to the model to analyze automatically the wide variety of reservoir geometries and operating policies encountered in practice.

An excellent description of this level-surface routing procedure (the Puls Method) is presented in Viessman et al. (2). Sound engineering judgement is essential in setting up this routing procedure. The input data and associated assumptions should be checked carefully.

Evaporation --

Evaporation losses are also accounted for in detention units. The loss is computed, at the end of each time step, by

$$e_v = A \cdot e_d \cdot \Delta t \qquad (5\text{-}8)$$

where e_v = evaporation loss, ft^3/time step,
A = surface area at the water level in the unit, ft^2,
e_d = evaporation rate, ft/sec., and
Δt = time step, sec.

The user must supply the values of e_d for each month of the simulation period. Losses are taken from the total storage volume present at the end of each time step. In the case where routing by plug flow is specified (see below) the loss is proportionally taken from each plug, depending on its proportion of the total storage volume.

Instantaneous Throughflow --

If the unit is specified to have no detention capability, then the model assumes that what arrives during a time step leaves as treated outflow that same time step less removed volumes. In this case, the volume of removed water is taken as a simple percentage of the inflow.

Pollutant Routing

Complete Mixing --

Pollutants are routed through a detention unit by one of two modes: Complete mixing or plug flow. For complete mixing, the concentration of the pollutant in the unit is assumed to be equal to the effluent concentration. The mass balance equation for the assumed well mixed, variable volume reservoir shown in Figure 5-3 is (3):

$$\frac{d(VC)}{dt} = O(t)\ C(t) - I(t)\ C^I(t) - K\ C(t)\ V(t) \qquad (5\text{-}9)$$

where V = reservoir volume, ft^3

C^I = influent pollutant concentration, mg/1,
C = effluent and reservoir pollutant concentration, mg/1,
I = inflow rate, ft^3/sec,
O = outflow rate, ft^3/sec,
t = time, sec and
K = decay coefficient, $sec.^{-1}$

Equation 5-9 is very difficult to work with directly. It may be approximated by writing the mass balance equation for the pollutant over the interval, Δt:

Change in mass in tank during Δt = Mass entering during Δt - Mass leaving during Δt - Decay during Δt

$$C_2V_2 - C_1V_1 = \frac{C_1^I\ I_1 + C_2^I\ I_2}{2}\ \Delta t - \frac{C_1O_1 + C_2O_2}{2}\ \Delta t - K\ \frac{C_1V_1 + C_2V_2}{2}\ \Delta t \qquad (5\text{-}10)$$

where subscripts 1 and 2 refer to the beginning and end of the time step, respectively.

From the reservoir routing, I_1, I_2, O_1, O_2, V_1, and V_2 are known. The concentration in the reservoir at the beginning of the time step, C_1, and the influent concentrations, C_1^I and C_2^I are also known as are the decay rate, K, and the time step, Δt. Thus, the only unknown, the end of period concentration, C_2, can be found directly by rearranging equation 5-10 to yield

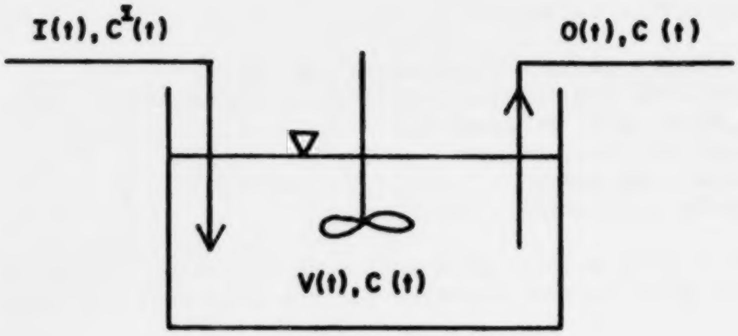
I(t), C^I(t)
O(t), C (t)
V(t), C (t)

Figure 5-3. Well-mixed, Variable Volume Reservoir (4).

$$C_2 = \frac{C_1V_1 + \frac{(C_1^I\ I_1 + C_2^I\ I_2)}{2}\ \Delta t - \frac{C_1O_1}{2}\ \Delta t - \frac{K\ C_1V_1}{2}\ \Delta t}{V_2(1 + \frac{K\ \Delta t}{2}) - \frac{O_2}{2}\ \Delta t} \qquad (5\text{-}11)$$

Equation 5-11 is the basis for the complete mixing model of pollutant routing through a detention unit.

Plug Flow --

If the user selects the plug flow option, the inflow during each time step, herein called a plug, is labeled and queued through the detention unit. Transfer of pollutants between plugs is not permitted. The outflow for any time step is comprised of the oldest plugs, and/or fractions thereof, present in the unit. This is accomplished by satisfying continuity for the present outflow volume (which was calculated earlier):

$$\sum_{j=JP}^{LP} V_j \cdot f_j = V_o \qquad (5\text{-}12)$$

where V_o = volume leaving unit during the present time step, ft^3,

V_j = volume entering unit during j^{th} time step (plug j), ft^3,

f_j = fraction of plug j that must be removed to satisfy continuity with V_o, $0 \leq f_j \leq 1$,

JP = time step number of the oldest plug in the unit, and
LP = time step number of the youngest plug required to satisfy continuity with V_o.

The detention time (sec) for each plug j is calculated as

$$(t_d)_j = (KKDT - j)\ \Delta t \qquad (5\text{-}13)$$

where KKDT = present time step number.

For a plug j leaving the unit, the amount of pollutant leaving is

$$(P_o)_j = (P_i)_j\ f_j\ (1.0 - R_j) \qquad (5\text{-}14)$$

where $(P_o)_j$ = amount of pollutant leaving unit in plug j, lbs,

$(P_i)_j$ = amount of pollutant entering unit with plug j, lbs, and

R_j = removal fraction for plug j

The manner in which R_j is calculated is decided by the user; however, as with the complete mixing option, R_j should be function of $(t_d)_j$. The technique for developing removal equations is discussed later. The remaining pollutants in each plug leaving the unit are totaled to give the total amount discharged during the present time step.

In either mode, removed pollutants are accumulated in the unit and exit with the removed water to form the residuals stream. The fraction of accumulated pollutants removed from the unit in any one time step is equal to the fraction of stored water removed from the unit. The water removal rate is specified by the user as a fraction of the remaining storage or a constant rate.

Instantaneous Throughflow --

Pollutants are routed instantaneously through units modeled as having no detention capability. In other words, the pollutants arriving during a time step leave the same time step less the removed portion. The removed pollutants are combined with the removed water to form the residuals stream. The rate at which water is removed is a user specified fraction of the inflow.

Sludge Handling --

Sludge (or sludge volume) is assumed to consist of the removed suspended solids and water from any storage/treatment unit. Sludge is not routed differently from any other flow in the system - it is simply one of the flows leaving a unit. In other words, the residuals stream from a unit can only be termed "sludge" if suspended solids are routed. The removal techniques discussed below are applicable to sludge handling units as well as mainline units.

Pollutant Characterization

Pollutants are characterized by their magnitude (i.e., mass flow and concentration) and, if the user desires, by particle size and specific gravity distributions. Describing pollutants by their particle size distribution is especially appropriate where small or large particles dominate or where several storage/treatment units are operated in series. For example, if the influent is primarily sand and grit, then a sedimentation unit would be very effective; if clay and silt predominate, sedimentation may be of little use. Also, if several units are operated in series, the first units will remove a certain range of particle sizes thus affecting the performance of downstream units. Therefore, the need for describing pollutants in more detail is obvious for modeling purposes. The pollutant removal mechanism peculiar to each characterization is discussed below.

Pollutant Removal

Characterized by Magnitude --

If pollutants are characterized only by their magnitude then the model improves the quality of the waste stream by removal equations. Removal of any pollutant may be simulated as a function of detention time (in minutes, detention units only), incoming concentration, inflow rate, the removal fraction of another pollutant, the incoming concentration of another pollutant or any combination of the above "removal factors". This selection is left to the user. Two functional forms are provided by the program to construct the desired removal equation:

$$R = a_9 e^{a_1 x_1} x_2^{a_2} + a_{10} e^{a_3 x_3} x_4^{a_4} + a_{11} e^{a_5 x_5} x_6^{a_6} + a_{12} e^{a_7 x_7} x_8^{a_8} \qquad (5\text{-}15)$$

or

$$R = a_9 e^{(a_1x_1 + a_2x_2 + a_3x_3 + a_4x_4)} x_5^{a_5} x_6^{a_6} x_7^{a_7} x_8^{a_8} \qquad (5\text{-}16)$$

where x_i = removal equation variables,
a_j = coefficients, and
R = removal fraction, $0 \leq R \leq 1.0$.

Within the program, the user may assign removal equation variables, x_i, to be parameters such as detention time, flow rate, inflow concentration of the parameter being removed, inflow concentration of another parameter, etc. These are seen to be parameters which are computed within the program at each time step. (If they are not assigned to be specific parameters, the remaining x_i are set equal to 1.0 for the duration of the simulation.) The coefficients, a_j, are directly specified by the user. There is considerable flexibility contained in these two forms, and with a judicious selection of coefficients and factors, the user can probably create the equation he desires. Example applications of equations 5-15 and 5-16 are given below to illustrate the procedure.

Earlier versions of the Storage/Treatment Block employed the following removal equation for suspended solids in the sedimentation unit (5):

$$R = R_{max}(1 - e^{-Kt_d}) \qquad (5\text{-}17)$$

where R = suspended solids removal fraction, $0 \leqq R \leqq R_{max}$,
R_{max} = maximum removal fraction,
t_d = detention time, min., and
K = first order decay coefficient, 1/min.

This same equation could be built from equation 5-15 by setting $a_9 = R_{max}$, $a_{10} = -R_{max}$, $a_3 = -K$ and letting x_3 = detention time, t_d. All other coefficients, a_j, would equal zero.

Another example is taken from a study by Lager et al. (6). Several curves for suspended solids removal from microstrainers were derived with a variety of aperture sizes. Fitting a power function to the curve representing a 35-micron microstrainer yields

$$R = 0.0963\ SS^{0.286} \qquad (5\text{-}18)$$

where R = suspended solids removal fraction, and $0 \leqq R \leqq 1.0$, and,
SS = influent suspended solids concentration, mg/l.

Equation 5-16 can be used to duplicate this removal equation by setting $a_9 = 0.0963$, $a_5 = 0.286$ and x_5 = influent suspended solids concentration, SS. All other a_j are zero.

Sludge handling may also be modeled with equations 5-15 and 5-16. Figure 5-4 shows the reduction in volatile solids in raw sludge (suspended solids - see earlier discussion) by a digester as a function of percent volatile solids and detention time (4). These curves can be approximated by

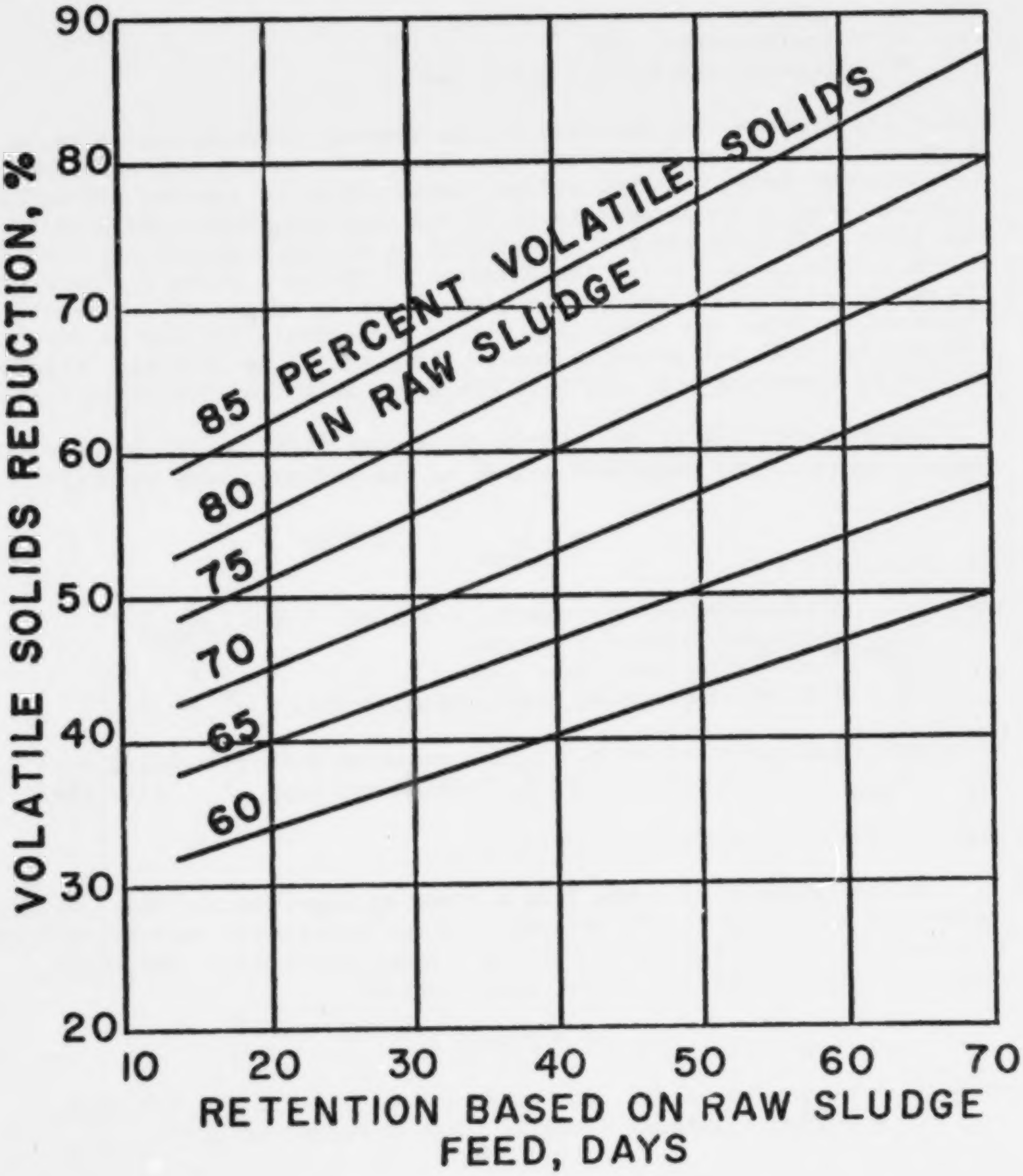

Figure 5-4. Reduction in Volatile Solids in Raw Sludge (4).

$$R = 1.31 \times 10^{-4} \left(\frac{t_d}{1440}\right)^{0.33} P_v^{1.67} \qquad (5\text{-}19)$$

where R = volatile solids reduction, $0 \leq R \leq 1.0$
t_d = detention time, min,
P_v = percent volatile solids in raw sludge,

$$P_V = 100 \frac{VS}{SS} \qquad (5\text{-}20)$$

where VS = influent volatile solids concentration, mg/l, and
SS = influent suspended solids (raw sludge) concentration, mg/l.

Equation 5-16 can be used to construct equation 5-19 by setting $a_9 = (1.31 \times 10^{-4})(1440)^{-0.33}$, $a_5 = 0.33$, $a_6 = 1.67$, $a_7 = -1.67$, x_5 = detention time, t_d, x_6 = influent volatile solids concentration, VS, and x_7 = influent suspended solids (raw sludge) concentration, SS. A current description of sludge handling can be found in references 7-9.

Characterized by Particle Size and Specific Gravity Distribution -- Distribution - If pollutants are characterized by their particle size and specific gravity distribution, then they are removed from the waste stream by particle settling or obstruction. Many storage/treatment processes use these physical methods to treat wastewater; sedimentation and screening are among the most obvious examples.

In this mode, pollutants are apportioned over several (up to 10) particle size/specific gravity ranges (e.g., ten percent of the BOD is found in the range from 10 to 50 microns). Each of the ranges is preset by the user and assigned an upper and lower bound on the particle diameter and a value for specific gravity. The user also specifies the apportionment of pollutants over the various ranges as they enter the first unit. This distribution is modified as it passes through the storage/treatment plant. Unfortunately, the distribution entered at the first unit must remain constant over time since the other blocks of SWMM do not provide a time-varying particle size distribution.

Each unit removes all or some portion of each range; the associated removal of pollutants is easily determined. For example, if a sedimentation unit removes 50 percent of the particles in the 50 to 100 micron range and ten percent of the pollutant in question is found in this range, then five percent of the total pollutant load is removed. The total removal is determined by summing the effects of several ranges passing through this unit. Once certain particles are removed, the distribution of particle sizes for the outflow can be determined and passed on to the next unit or receiving water. The removed particles constitute the size distribution for the residuals stream. The next several paragraphs describe the two mechanisms available to the user for pollutant removal when pollutants are characterized by particle size.

Particle settling - There are several forms of settling: unhindered settling by discrete particles, settling by flocculating particles, and hindered settling by closely spaced particles (10). For simplicity, the unhindered settling of discrete particles will be the removal mechanism simulated in this model.

Discrete particles settling in a quiescent fluid accelerate to the point where the drag force exerted by the suspending fluid reaches equilibrium with the gravitational force exerted on the particle (10). At this point, the particle settles at a constant velocity known as the terminal velocity. By equating the forces acting on such a particle, the equation for the terminal or settling velocity of the particle is derived and approximated by (10)

$$v_s = \sqrt{\frac{4}{3} \frac{g}{C_D} (S_p - 1) d} \qquad (5\text{-}21)$$

where v_s = terminal velocity of particle, ft/sec,
g = gravitational constant, 32 ft/sec^2,
C_D = drag coefficient,
S_p = specific gravity of particle, and
d = diameter of particle, ft.

Additionally,

$$C_D = \frac{24}{N_R} \text{ , if } N < 0.5\text{, or} \qquad (5\text{-}22)$$

$$C_D = \frac{24}{N_R} + \frac{3}{\sqrt{N_R}} + 0.34\text{, if } 0.5 \leqq N_R < 10^4\text{, or,} \qquad (5\text{-}23)$$

$$C_D \cong 0.4\text{, if } N_R \geqq 10^4. \qquad (5\text{-}24)$$

where N_R = Reyonlds number, dimensionless,

$$N_R = v_s \, d/v \qquad (5\text{-}25)$$

and v = kinematic viscosity, ft^2/sec.

The procedure for finding v_s under any of the above conditions is demonstrated by Sonnen (11) and is computed at the beginning of the simulation and stored for later use. The average of the high and low ends of each particle size range is used as the representative particle size for use in the above calculations.

A range of conditions may exist in an actual detention unit, from very quiescent, to highly turbulent and nonquiescent. Camp's (12) ideal removal efficiency, E_Q, will be used for quiescent conditions, and an adaptation of his sedimentation trap efficiency curves (12, 13, 14) as described by Chen (15) will be used to make the extension to nonquiescent conditions, as described below.

For quiescent conditions,

$$E_Q = \min \begin{cases} 1 \\ v_s/v_u \end{cases} \qquad (5\text{-}26)$$

where E_Q = particle removal efficiency as a fraction, $0 \leq E_Q \leq 1$,

v_s = terminal velocity of particle, ft/sec, and
v_u = overflow velocity, ft/sec.

Additionally,

$$v_u = Q/A = \frac{Ay/t_d}{A} = y/t_d \qquad (5\text{-}27)$$

where Q = flow rate, ft^3/sec,
A = surface area of detention unit, ft^2,
y = depth of water in unit, ft, and
t_d = detention time, sec.

Equation 5-27 assumes a rectangular detention unit with vertical sides. However, a circular unit (with vertical sides) may also be modeled when characterizing pollutants by particle size. In other words, equation 5-27 is restricted to units that allow the surface area to remain constant at any depth. Applying this equation (and, thus, the entire particle size methodology) to other unit types should only be done when the surface area is independent of depth.

Equation 5-26 represents an ideal quiescent basin in which all particles with settling velocities greater than v_u will be removed. Deviations from quiescent conditions can be handled explicitly based on Camp's (12) sedimentation trap efficiency curves, which were developed as a complex function of particle settling velocity and several basin parameters.

$$E = f\left(\frac{v_s y}{2\varepsilon} , \frac{v_s A}{Q} = \frac{v_s \ell}{v_t y} = \frac{v_s}{v_u} \right) \qquad (5\text{-}28)$$

where E = particle removal efficiency, $0 \leqq E \leqq 1$,
ε = vertical turbulent diffusivity or mixing coefficient, ft^2/sec,
v_t = flow through velocity of detention unit, ft/sec,
ℓ = travel length of detention unit, ft, and
other terms are defined previously.

Camp (12) solves for the functional form of equation 5-28 assuming a uniform horizontal velocity distribution and constant diffusivity, ε. A form of the advective-diffusion equation then results in which local changes in concentration at any vertical elevation are equal to the net effect of settling from above and diffusion from below. The diffusivity will be constant if the horizontal velocity is assumed to have a parabolic distribution, (although this assumption is clearly at variance with the uniform velocity distribution assumption above). For the parabolic distribution, ε is then found from

$$\varepsilon = 0.075\ y\sqrt{\tau_o/\rho} \qquad (5\text{-}29)$$

where τ_o = boundary shear stress, lb/ft^2 and
ρ = density of water ≅ 1.94 slug/ft^3.

The term $\sqrt{\tau_o/\rho}$ is known as the shear velocity, u_*, and can be evaluated using Manning's equation for open channel flow (14),

$$u_* = \sqrt{\tau o/\rho} = \frac{v_t n\sqrt{g}}{1.49\ y^{1/6}} \qquad (5\text{-}30)$$

where n = Manning's roughness coefficient.

The flow through ("horizontal") velocity, v_t, is also given by

$$v_t = \ell/t_d \qquad (5\text{-}31)$$

where ℓ = travel length of detention unit, ft, and
t_d = detention time, sec.

Equations 5-29 and 5-30 are then used to convert $v_s y/2\varepsilon$ to a more useable form,

$$\alpha = 0.1\frac{v_s y}{2\varepsilon} = \frac{v_s}{1.5u_*} = \frac{v_s y^{1/6}}{v_t\ n\sqrt{g}} \qquad (5\text{-}32)$$

where α = turbulence factor, dimensionless when all parameters are in units of feet and seconds.

Camp's sedimentation trap efficiency curves (12 - 15) are the solution to the advective-diffusion equation mentioned previously and are shown in Figure 5-5 as a function of α. Ideally, these curves could be included in the model in some manner, but their representation is not straight forward from a programming standpoint. Instead, a simplification is used, based on early work of Hazen (16) and the Bureau of Reclamation as described by Chen (15).

It is assumed that an upper limit on turbulent conditions is given by $\alpha = 0.01$. Removal efficiency under these conditions is accurately represented by the function fitted to the ordinate of Figure 5-5,

$$E_t = (1 - e^{-v_s/v_u}) \qquad (5\text{-}33)$$

or

$$E_t = (1 - e^{-v_s\ t_d/y}) \qquad (5\text{-}34)$$

where E_t = particle removal efficiency under turbulent conditions, $0 \le E_t \le 1$.

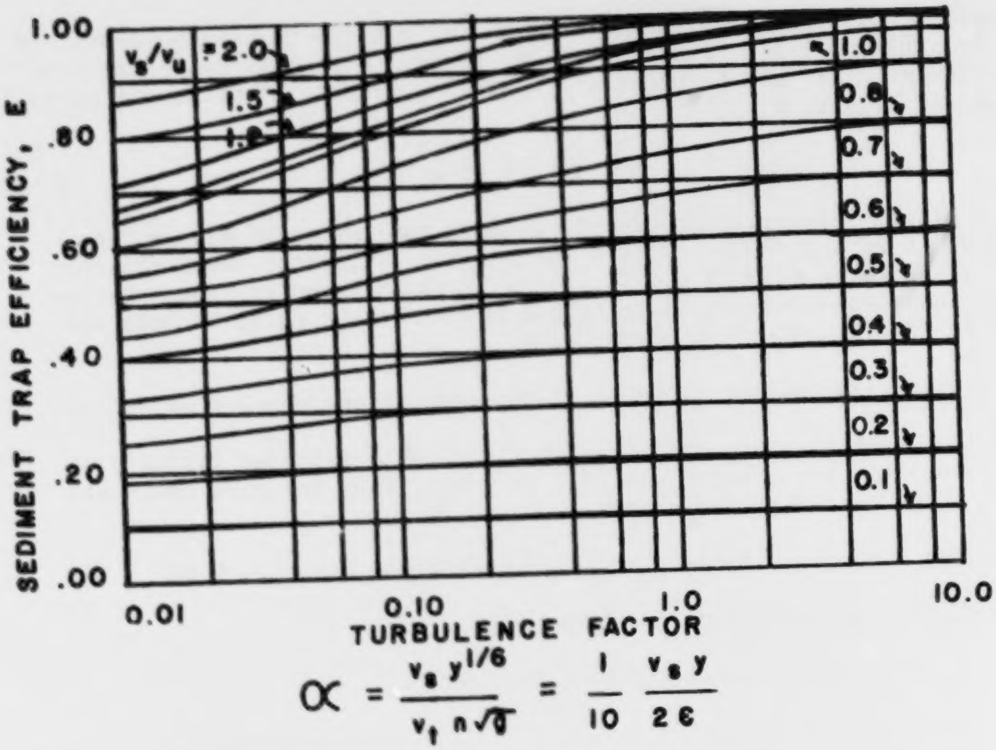

Figure 5-5. Camp's Sediment Trap Efficiency Curves (12-15).

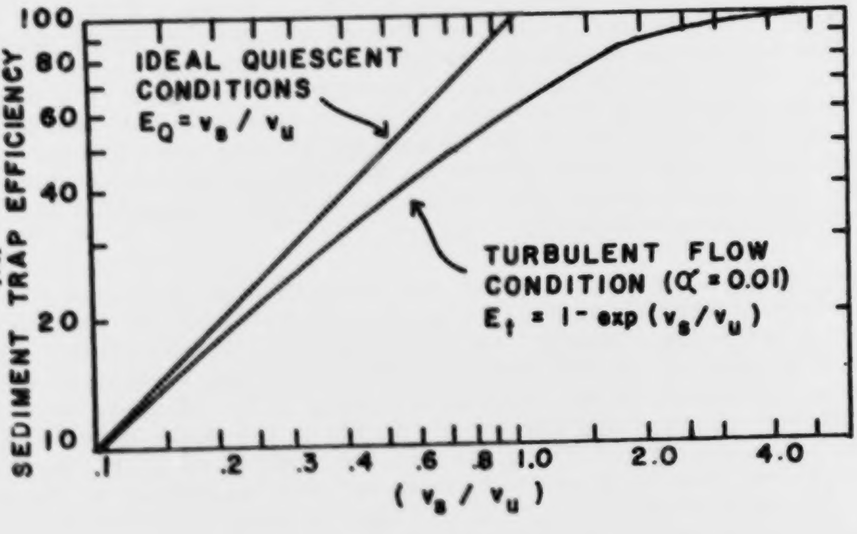

Figure 5-6. Limiting Cases in Sediment Trap Efficiency (15).

Quiescent conditions are assumed to exist for $\alpha = 1.0$ for which removal is given by equation 5-26. Equations 5-26 and 5-33 are shown in Figure 5-6. The parameter α may now be used as a weighting factor to obtain the overall removal efficiency, E,

$$E = \alpha E_Q + (1-\alpha) E_t \tag{5-35}$$

Thus, a linear interpolation is made between the curves of Figure 5-6 (equations 5-26 and 5-33). Within the program, values of the turbulence factor are limited to $0.01 \leqq \alpha \leqq 1.0$. If a value computed from equation 5-32 is less than 0.01 and (more turbulent) it is set equal to 0.01 and similarly for the quiescent boundary.

To summarize, the particle settling computations proceed as follows.

1. For each size and specific gravity range a settling velocity is computed using equations 5-21 to 5-25. Then for each range all steps below are performed.
2. The turbulence factor, α, is computed from equation 5-32.
3. E_Q is computed using equation 5-26.
4. E_t is computed using equation 5-33 or 5-34.
5. Finally, the removal efficiency for the particular particle size and specific gravity range is computed from equation 5-35.

In a normal simulation, several plugs leave the detention unit in any given time step. The effluent is all or part of a number of plugs depending on the required outflow as determined by the storage routing techniques discussed earlier. Thus, the effluent particle size distribution is a composite of several plugs. This composite distribution is determined by taking a weighted average (by pollutant weight in each plug) over the effluent plugs. This distribution is then routed downstream for release or further treatment. The particles that were removed from each plug are also composited and are combined with the sludge volume.

Particle obstruction - The second major removal mechanism used when pollutants are characterized by particle size distributions is the obstruction or screening of particles. Removal efficiencies are a result of two actions: the straining of the screens, and the additional filtration provided by the mat produced by the initial screening (17). Screens vary widely in the size of the aperture and the manner in which the waste water flows through them. To simplify the calculations, the removal of particles is assumed to be a function of screen size only and the filtration by the mat is ignored. In other words, a particular screen size will remove only those particles larger than that size. This is not entirely accurate, of course, but the result is a conservative removal estimate that may be accurate in cases where backwashing is at a relatively high rate. In fact, a study by Maher indicates that this simplifying assumption is reasonable (17). In this case, a microstrainer with a Mard "O" screen (aperture of 23 microns) was installed in a residential area of Philadelphia, Pennsylvania. The analysis of the backwash material for two storms (one in which a coagulant was used) revealed that, by particle count, 88 to 96 percent of the particles were indeed smaller than 23 microns. However, by weight, over 99 percent of the material was found in particles greater than 23 microns. Although Maher did not report the distribution in terms of weight, it was a simple matter to convert by assuming a specific gravity.

During the simulation, a screen alters the particle size distribution for a particular time step without detention time. Again, only the particles larger than the screen aperture are removed. If the screen size falls between the high and low ends of any range, the pollutants are removed by simple linear interpolation. For example, if 20 percent of the suspended solids are found in the range from 10 to 50 microns and the screen aperture is 20 microns, then 75 percent of the suspended solids in that range will be removed or 15 percent of the total suspended solids load. Of course, if the entire range is larger than the size of the aperture, then all pollutants in that range are removed.

Comment on characterization by particle size distribution - Pollutants characterized by a particle size distribution are restricted by the model to the two removal mechanisms discussed above. This limits the user somewhat if this characterization is chosen. The types of units that could be considered in this case would include sedimentation tanks, regular storage basins, bar racks, fine screens, microscreens and swirl concentrators. However, these units probably represent the bulk of the processes applied to the problem of combined sewer overflow and stormwater runoff. Thus, limits of the applicability of the model using this mode are probably not too severe.

Cost Calculations

Initial capital and operation and maintenance costs are calculated at the end of a simulation. These costs are computed using only the information processed for the simulation period. In other words, no attempt is made to derive costs for particular time intervals (e.g., annual). It is left for the user to interpret the results produced by the subroutine STCOST.

The capital cost for each unit is computed as a function of a design flow or volume specified by the user or is calculated by the model as a function of the maximum value recorded during the simulation.

$$C_{cap} = a\ Q^{b}_{max} \qquad (5\text{-}36)$$

or

$$C_{cap} = a\ (Q_{in})^{b}_{max} \qquad (5\text{-}37)$$

or

$$C_{cap} = a\ V^{b}_{max} \qquad (5\text{-}38)$$

or

$$C_{cap} = a\ (V_{obs})^{b}_{max} \qquad (5\text{-}39)$$

where C_{cap} = initial capital cost, dollars,

Q_{max} = maximum allowable inflow, ft^3/sec,

$(Q_{in})_{max}$ = maximum inflow encountered during the simulation, ft^3/sec,

V_{max} = maximum allowable storage (detention units only), ft^3,

$(V_{obs})_{max}$ = maximum storage encountered during the simulation, ft^3, and

a, b = coefficients (specified by the user).

Power functions are frequently used in wastewater treatment cost estimations. Therefore, the above equations should be widely applicable.

Operation and maintenance costs are calculated as functions of the variables listed above and the total operating time (calculated as the number of time steps with inflow to the unit).

$$C_{om} = d\, Q_{max}^{f} + gD_{op} \tag{5-40}$$

or

$$C_{om} = d(Q_{in})_{max}^{f} + gD_{op} \tag{5-41}$$

or

$$C_{om} = d\, V_{max}^{f} + gD_{op} \tag{5-42}$$

or

$$C_{om} = d(V_{obs})_{max}^{f} + gD_{op} \tag{5-43}$$

where D_{op} = total operating time during the simulation period, hours, and

d,f,g = coefficients (supplied by the user).

The user is cautioned not to misinterpret the cost calculated by the model. For example, in a single-event simulation the calculated capital cost could only be considered an estimator of the true capital cost when the event simulated is a design event. Likewise, when operating time is a factor in computing operation and maintenance costs, the calculated costs can be a valid estimator of the true costs only when a long term simulation is performed. Recent EPA publications provide useful information for the proper selection of the coefficients required in equations 5-36 through 5-43 (18, 19).

SUMMARY

A new Storage/Treatment Block has been developed that is somewhat different from its predecessor. The model requires greater user input and knowledge of the processes being modeled. Storage/Treatment units may be modeled as detention or non-detention units. Pollutants may be characterized by their magnitude alone or by magnitude and their particle size/specific gravity distribution. Any three of the pollutants available from other blocks may be routed through the S/T Block. A simple cost routine is also included.

In summary, the Storage/Treatment Block offers the user a flexible tool for modeling wet- and dry-weather facilities and evaluating their performance and costs.

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ON-SITE CONTROL OF NONPOINT SOURCE POLLUTION

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INTRODUCTION

It is widely recognized that nonpoint source (NPS) pollution is a major problem in both urban and rural areas. Table 1 provides some typical levels for eight pollutants from 3 land uses. Considerable effort has been expended in developing stormwater management programs, with a majority of the resources being spent on evaluating regional effects and control. Much of this effort has involved model development for the purpose of predicting regional pollution levels as a function of land use change. However, little effort has been expended for analyzing the potential of on-site control as a means of reducing NPS loadings. Two methods that have significant potential for controlling NPS pollution in urban areas are street sweeping and detention storage.

THE USE OF STREET SWEEPING FOR CONTROL OF NONPOINT SOURCE POLLUTION

The recent concern about nonpoint source pollution in runoff from urban and suburban areas should focus attention on the effectiveness of street sweeping as a means of controlling nonpoint source pollution. Unfortunately, past investigations on the efficiency of street sweeping are few and primarily concerned about control of either radioactive fallout or the dust and dirt fraction of street wastes. And the variation in the results is important. Studies report removal fractions from 0.1 to 0.95, with the wide variation being due to factors such as differences in particle size distribution, sweeper and broom type, sweeper speed, and pavement type and condition.

Because of the importance of street sweeping in controlling nonpoint source pollution, the modeling approach appears to provide a convenient means of examining the efficiency of street sweeping in controlling nonpoint source pollution. A model was developed (8) that included three components:

1) algorithms describing the accumulation of pollutants on residential, commercial, and industrial land uses; 2) a hydrologic component that describes the removal of pollutants due to storm water runoff; and 3) a component to estimate the removal of pollutants from streets by street sweepers. Accumulation of pollutants are defined in the model to be a function of time between storm events or street sweepings, land use, pavement type and condition, population density, and traffic density. The removal of pollutants by surface runoff is a function of the initial pollutant loading estimated from accumulation component, the total volume of rainfall, the percentage of impervious area in the watershed, the slope of the gutter or surface, and the overland flow length. The effectiveness of street sweeping in removing pollutants is developed as a function of the characteristics of the initial pollutant loading, the forward speed of the street sweeper, the type of street sweeper, and the pavement type. The model can simulate the following pollutants: total solids, volatile solids, 5-day biochemical oxygen demand, chemical oxygen demand, Kjeldahl nitrogen, nitrates, phosphates, and total heavy metals.

The model was designed as a planning model to be used for a variety of common planning problems, such as the evaluation of alternative corrective measures, evaluation of alternative growth plans, the evaluation of the effectiveness of public works scheduling, and the evaluation of the cost effectiveness of policy constraints (4,9).

To measure the sweeping effectiveness of a sweeper a concept of relative effort RE has been defined as:

$$RE = \frac{1200 \text{ ft/min}}{\text{forward speed of sweeper in ft/min}} \qquad (1)$$

Using data from in situ tests reported in the literature (7) equations that relate the mass remaining to relative effort were calibrated from selected particle size ranges, pavement type (concrete and asphalt), and sweeper type (motorized and vacuumized).

To demonstrate the flexibility and validity of the model, field surveys were conducted at four specific locations within Washington, D.C. The test sites included an industrial site, a commercial area, multi-family housing, and single family land use. Also, sites were selected so that the model was tested under a variety of important site characteristics.

Table 2 shows the percentage removal of eight pollutants for the test site in single-family residential land use, with the results for sweeping frequencies of 2, 4 and 7 days, forward speeds of 3 mph and 4 pmh, and both vacuumized and motorized sweepers. Table 2 indicates that the frequency of sweeping was the most important variable, with sweeper type and forward speed being considerably less important. For lower speeds the relative effort is greater and thus the removal percentages are slightly higher. The highest removal efficiency was for total solids while phosphates had the lowest removal efficiency. The results suggest that sweepers are not highly efficient in removing pollutants from urban streets. This stems from the inability of the sweepers to remove the smaller particles, representing the greater bulk of the pollutants.

While removal percentages may be low, the alternative is to have the pollutants washed off during storm events. Table 1 shows the magnitude of the problem. Values such as these indicate that nonpoint source pollution is a serious problem and failure to sweep will lead to either serious degradation of receiving streams or increased demand for treatment of storm water runoff.

THE USE OF ON-SITE DETENTION FOR CONTROL OF NONPOINT SOURCE POLLUTION

It should be evident that street sweeping cannot serve as an independent means of NPS pollution control. First, with average removal fractions less than 0.5, there is still a significant amount of pollutants available for removal by stormwater runoff. Second, while sweepers can be used in highly developed urban areas, there still remains a considerable contributing area where sweepers either are not an economical means of pollution control or cannot effectively service the source. Third, because of the importance of sweeping frequency (Table 2), the limitation of sweeper availability, and the random nature of runoff generating storms, it is not always possible to sweep an area before the pollutants are removed by runoff. Thus, sweeping must be considered as just one component of a water pollution control program.

Recognizing that stormwater detention is effective in controlling both flow rates and sediment, it should also be of interest in examining the impact of detention basins for water pollution control. The effectiveness of basins for pollution control will help determine the most cost-effective policies for stormwater management. By controlling pollution loadings at the source, it will reduce both water treatment costs and the degradation of water quality in streams, rivers, and estuaries.

The study site is a 148-acre urban subwatershed of Cabin John Creek in southcentral Montgomery County. The subwatershed includes all of the virtually total impervious Montgomery Mall shopping center, several moderate and high-rise apartment complexes, portions of a major highway and two secondary roadways, and two townhouse developments still under construction during the study. Some intervening green space remains among the several developments. Drainage from these areas outfalls into Montgomery Mall Lake at six different points. The major inflow point is at the head (east end) of the lake, where a 66-inch diameter RCP drains the shopping mall and 64 percent of the total area (95 acres) tributary to the lake.

The lake is about 750 feet long and 350 feet wide and has a 5.9 acre permanent pool, which at normal level varies from 3 to 13 feet in depth, allowing 37 acre-feet of dead storage capacity. Primary outflow is controlled by a 24-inch diameter CMP riser, topped with a 36-inch diameter circumferential hood, and connected to an 18-inch diameter horizontal CMP barrel. A 40-foot wide grassed overflow channel has a crest at an elevation 3 feet higher than the riser crest.

In addition to precipitation and flow rates both into and from Montgomery Mall Lake, the eleven water quality parameters were sampled during several storm events. Table 3 provides a summary of the characteristics of the water quality parameters.

The efficiency of a stormwater management basin for the control of pollutants can be measured using the trap efficiency TE:

$$TE = 1 - V_o/V_i$$

where V_o and V_i are the total volumes of a pollutant in the outflow and inflow, respectively. Table 3 provides a summary of trap efficiencies computed from median values of pollution levels in the observed data from the study site. While these values are based on median values, the values suggest that trap efficiencies can be quite high if the basin is designed for both flow and pollution control. Design parameters for pollution control are somewhat different than for flow control (3). The high values shown in Table 3 result partially from the large storage volume provided and the low return periods of the storms that produced the runoff. McCuen (6) provided relationships between the inflow and outflow rates of a detention structure and the levels of the pollutants in the inflow and outflow. The trap efficiency is easily computed from these estimates.

CONCLUSIONS

In a recent statement concerning policy recommendations in water resources, Ackermann, et al. (1), concluded,

> "In order to mitigate the effects of pollution, the following policies and procedures are recommended:
>
> 1. An intensive assessment of nonpoint sources of pollution should be obtained and sound and implementable drainage and land use management practices should be developed that will control them before they enter our surface water and ground water."

Davis, et al. (3), discussed one such intensive effort into assessing nonpoint source pollution, including a comprehensive data collection program. The study by Davis, et al., was one of the few studies that considered NPS pollution at the source in the upland areas of a watershed.

While considerable effort has been expended on estimating NPS pollution at the regional level, comparative little effort has been expended into NPS control on the local level. The results of this study show that stormwater management can be an effective means for controlling NPS pollution. This is especially significant because many state and local governments require detention storage for the control of both peak dischange and sediment in urbanizing areas. However, the design of detention basins should be different for flow control than it would be for pollution control, and thus, research on the multi-purpose design of detention facilities for both flow and quality control is needed. An advantage of controlling NPS at the local level is that water quality in both rivers and small streams should improve, whereas regional control measures may not result in the improvement of the water quality of small streams.

ACKNOWLEDGEMENT

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Table 1. Mean Loadings of Selected Nonpoint Source Pollutants*

Pollutant (lb/curb mile)	Residential	Industrial	Commercial
Total Solids	1200	2800	360
Volitile Solids	86	150	28
BOD_5	11	21	3
COD	25	100	7
Kjeldahl Nitrogen	2.0	3.9	0.4
Nitrates	0.06	0.18	0.18
Phosphates	1.1	3.4	0.3
Total Heavy Metals	0.58	0.76	0.18

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* Abstracted from Ref. 1

Table 2. Percentage Removal of Pollutants: Single-Family Residential Land Use

			Foward Speed of Street Sweeper					
			3 mph			4 mph		
Type of Sweeper	Pollutant	Frequency of Sweeping	2 Days	4 Days	7 Days	2 Days	4 Days	7 Days
Vacuumized	Total Solids		70	53	35	68	52	35
	Volatile Solids		55	40	26	53	39	26
	BOD		53	38	25	51	37	25
	COD		38	22	13	34	20	12
	Kjeldahl Nitrogen		55	39	25	53	38	25
	Nitrates		42	27	16	38	25	16
	Phosphates		25	11	6	20	9	5
	Total Heavy Metals		62	46	31	60	45	30
Motorized	Total Solids		69	52	34	67	51	34
	Volatile Solids		55	39	26	52	38	25
	BOD		53	37	25	50	36	24
	COD		39	22	13	35	21	12
	Kjeldahl Nitrogen		55	39	25	52	37	24
	Nitrates		42	27	16	39	25	16
	Phosphates		27	12	6	22	10	5
	Total Heavy Metals		62	46	30	59	44	29

TABLE 3. Water Quality Trap Efficiencies

Parameter	Units	Median Inflow Rate	Median Outflow Rate	Trap Efficiency
5-day BOD	10^{-3}lb/sec	11.35	0.37	.88
20-day BOD	10^{-3}lb/sec	19.7	1.35	.93
Chemical Oxygen Demand	10^{-3}lb/sec	60.5	2.10	.97
Zinc	10^{-6}lb/sec	0.463	0.0026	.99
Cadmium	10^{-6}lb/sec	0.36	0.0063	.98
Orthophosphates	10^{-3}lb/sec	0.308	0.022	.93
Total Organic Carbon	10^{-3}lb/sec	13.25	0.48	.96
Total Phosphorus	10^{-3}lb/sec	0.73	0.007	.99
Ammonia	10^{-3}lb/sec	6.8	0.025	.99
Lead	10^{-6}lb/sec	0.43	0.018	.96
Iron	10^{-6}lb/sec	10.3	0.373	.96

STORMWATER MODELLING APPLICATIONS[1.]

by Alan R. Perks[2.]

INTRODUCTION

In this brief paper, I would like to describe the background and framework within which much of our stormwater modelling is carried out in Ontario, and then review several different modelling applications recently carried out by Proctor & Redfern Limited.

Under the Canada-Ontario agreement for Great Lakes Water Quality, the Urban Drainage Subcommittee(1) was constituted to promote and oversee research into the problems of urban drainage. This committee adopted 3 basic objectives:

1. to define the problems
2. to develop the capabilities for solution
3. to develop a strategy for implementing solutions

Examples of the projects carried out under each of these objectives were monitoring studies,(2) Storm Water Management Model development,(3) and more recently preparation of provincial policies for urban drainage.(4) The adoption of these policies in the near future will signify the end of the subcommittee's program.

At present, however, the regulatory responsibility for urban drainage remains somewhat divided between different levels of government and different agencies. At the provincial level, the Ministry of the Environment is primarily concerned about water quality, whereas the Ministry of Natural Resources deals mainly with water quantity management. The design and construction of storm sewer systems and small channels is controlled primarily at the municipal level. There has fortunately been a vigorous interest in stormwater management on the part of all 3 agencies. In fact, several Ontario Municipalities have been among the first in Canada to adopt

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[1.] A paper presented to the 3rd Canadian-USA SWMM User's Group Meeting, Annapolis, Md., November, 1978.

[2.] Head of the Hydrotechnical Group, Proctor & Redfern Ltd. Toronto, Ontario.

new drainage criteria incorporating stormwater management principles and modelling concepts.

APPLICATIONS

Urban Stormwater models have been used extensively by Proctor & Redfern in such cities as Toronto, Hamilton, Mississauga, St. Catharines, London and Sault Ste. Marie. Generally, these applications related to the analysis of existing system deficiencies, the design of expansions to existing systems, the design of stormwater management systems for entirely new developments, or the study of entire small watersheds for drainage planning purposes. Our approach has been to utilize a set, or hierarchy of models ranging from simple to quite complex, and including STORM,(5) HYMO,(6) ILLUDAS,(7) and SWMM(8). Such a package of models affords the user the capability to analyze most urban drainage problems. Fig. 1 shows the typical uses to which these models may be put.

The following section briefly outlines several of our recent applications which are indicative of the range of problems encountered.

West St. Catharines

West St. Catharines is served by several small combined sewer systems which have a history of frequent flooding. An investigation of one of these areas was launched to develop a flooding-relief program.

The SWMM was applied to this 175 acre area and was calibrated to observed flooding instances and levels. An improved version of the EXTRAN routine was applied to simulate surcharging. The results for both the recorded storm and the 5 year design storm indicated a general inadequacy of all pipes in the system and extensive flooding.

Because both trunks and lateral sewers were responsible for flooding, alternatives for providing relief were limited to complete separation or a program involving trunk replacement combined with several stormwater management measures.

This latter alternative appeared most promising and was investigated more fully. Cost savings could be achieved by utilizing the existing lateral sewers and replacing the trunk only. Detailed simulations of particular blocks were carried out to determine surface hydrographs, the effects of roof leader disconnection and inlet controls at catchbasins, and the depth of gutter flows along the street. The results indicated that a program of encouraging surface flows could reduce the need to replace existing lateral sewers without causing excessive inconvenience. The new trunk sewer would have adequate capacity to accept both lateral sewer and surface gutter inflows. Fig. 2 shows the project area and some pertinent results.

The capability of the SWMM to model surface flows, pipe flows, and surcharge conditions was indispensable to the analysis of this system.

Idlewood Subdivision

The Idlewood Subdivision is a 185 acre development in Kitchener, Ontario. Located in a small natural watershed, there was concern over the downstream impacts of this development. Stormwater management measures were required to provide this watershed protection.

The subdivision featured an extensive recreational greenway system which was also planned to serve as a stormwater facility. Street and lot runoff would be directed to the greenways where it would be conducted to detention areas via grassed swales. The detention areas would ultimately discharge slowly to the creek.

The ILLUDAS model was used to develop design runoff hydrographs and to determine optimum sizes and locations for detention. It was demonstrated that the 5 year post-development flow would be approximately the same as under natural conditions. The ability of the model to handle pipes, channels, and detention storage was the main reason for its use in this case. Fig. 3 shows the drainage features of this project.

Kidd's Creek

Kidd's Creek is a small partly-urban watershed located in Barrie, Ontario. The downstream section is in an older residential area and is quite congested for drainage. Flooding occurs here frequently. The upper two-thirds of the watershed is undergoing rapid development, and concern has been expressed about the effects of this development on downstream flooding. Fig. 4 shows a schematic of the Kidd's Creek watershed.

HYMO was used to determine watershed flows at various points, and these were then used to establish inadequacies in the downstream culverts and channels. Floodlines were determined by water surface profile calculations. On this basis, several programs of remedial works were developed for different design frequencies.

It appeared, however, that stormwater management practices in the upstream developments would reduce the need for expensive downstream improvements. Further simulations of drainage conditions and storages confirmed the feasibility of this approach, and the final plan recommended rigorous stormwater management coupled with minimum downstream improvements to satisfy existing flows.

Further detailed SWMM and ILLUDAS modelling will be required at the design stage to establish effective stormwater management schemes for each area.

CONCLUSIONS

Modelling techniques aid the understanding and development of complex stormwater management schemes, and provide the designer greater capacity to evaluate alternatives. Our firm has found them useful in a great variety of projects, typified by the examples cited above.

One of the more interesting areas of modelling applications is in the implementation of stormwater management concepts in new developments. In Southern Ontario, a great deal of development is planned over the next decade which will significantly influence our urban watersheds.

The competing demands for environmental protection and maximum land usage and yields creates conditions where a great number of alternatives must be thoroughly analyzed before a master plan can be adopted.

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FIGURES

Figure 1 - Use of Different Runoff Models

Figure 2 - West St. Catharines Project Area

Figure 3 - Idlewood Subdivision Drainage Features

Figure 4 - Schematic of Kidd's Creek Watershed

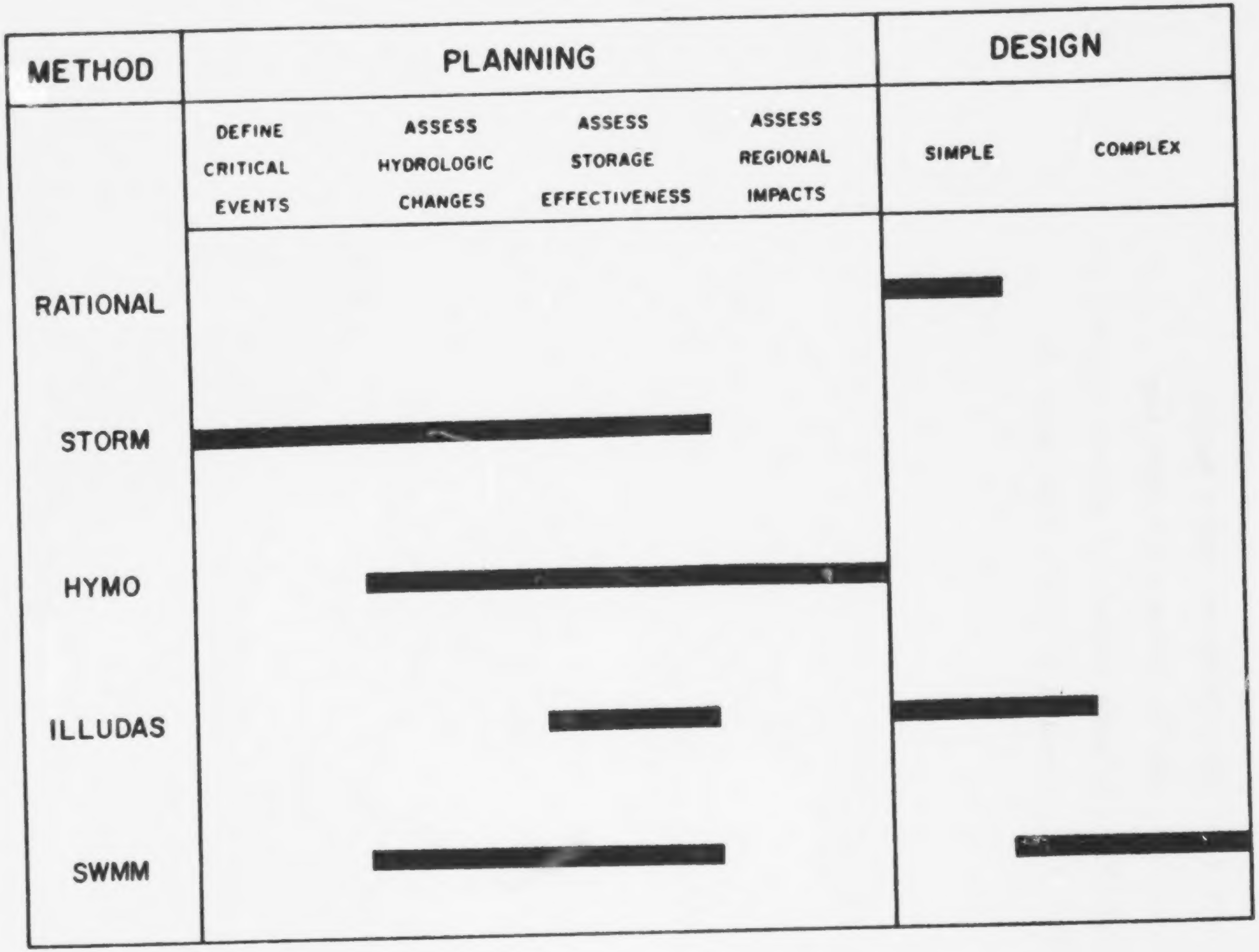
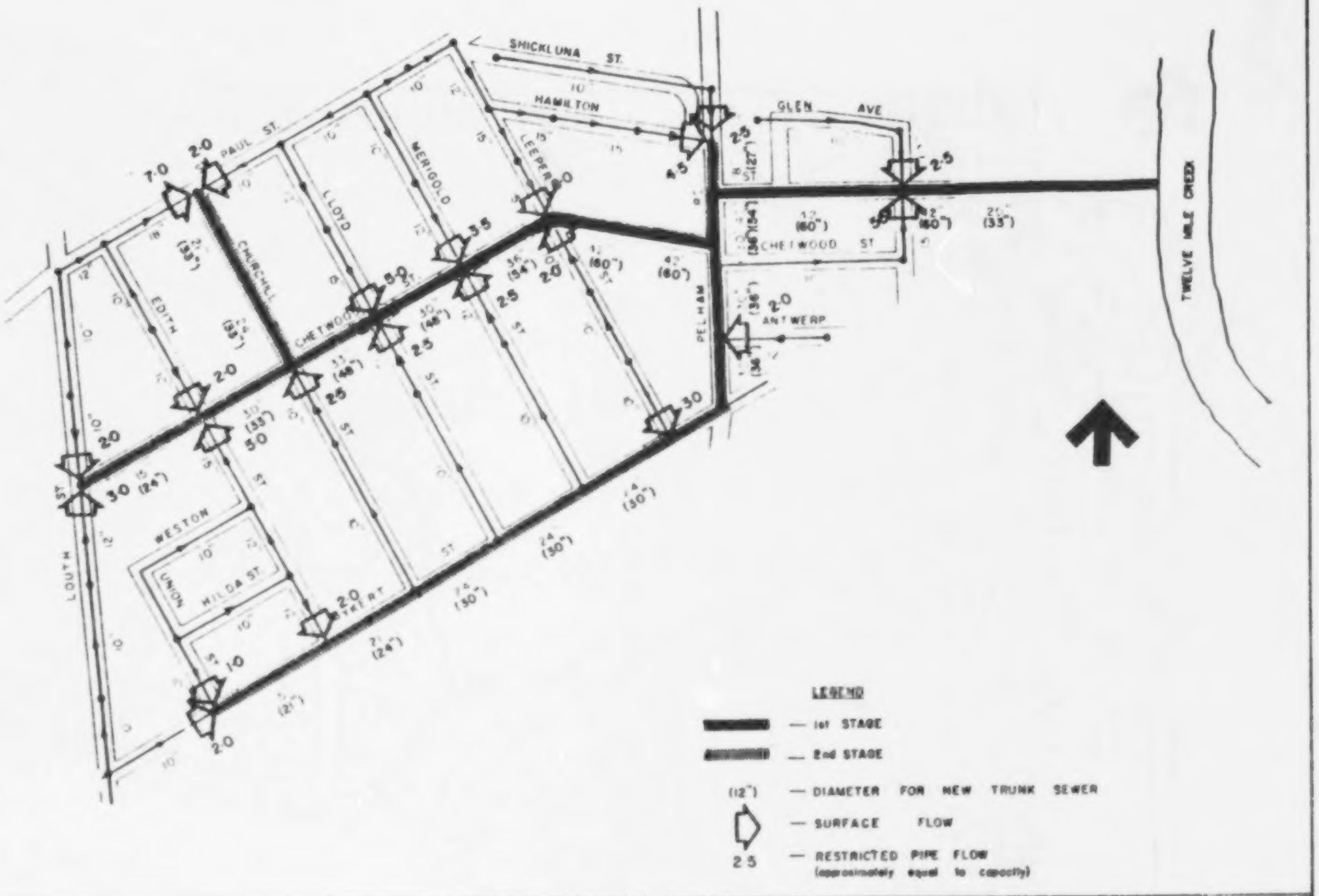


FIGURE 1(9) - USE OF DIFFERENT RUNOFF METHODS

Area 29


TWELVE MILE CREEK
SHICKLUNA ST.
HAMILTON
GLEN AVE
PAUL ST.
EDITH
CHURCHILL
LLOYD
MERIGOLD
LEEPER
CHETWOOD ST
CHETWOOD
PELHAM
ANTWERP
WESTON
UNION
HILDA ST.
TALBOT
LOUTH
ST
LEGEND
— 1st STAGE
— 2nd STAGE
(12") — DIAMETER FOR NEW TRUNK SEWER
— SURFACE FLOW
2.5 — RESTRICTED PIPE FLOW (approximately equal to capacity)

ALTERNATIVE RELIEF 3 - Temporary Street Storage
- Main Trunk Upgraded for 5 Year Level
- 60% Roof Disconnection

FIGURE 2

WEST ST. CATHERINES
PROJECT AREA

Figure 3

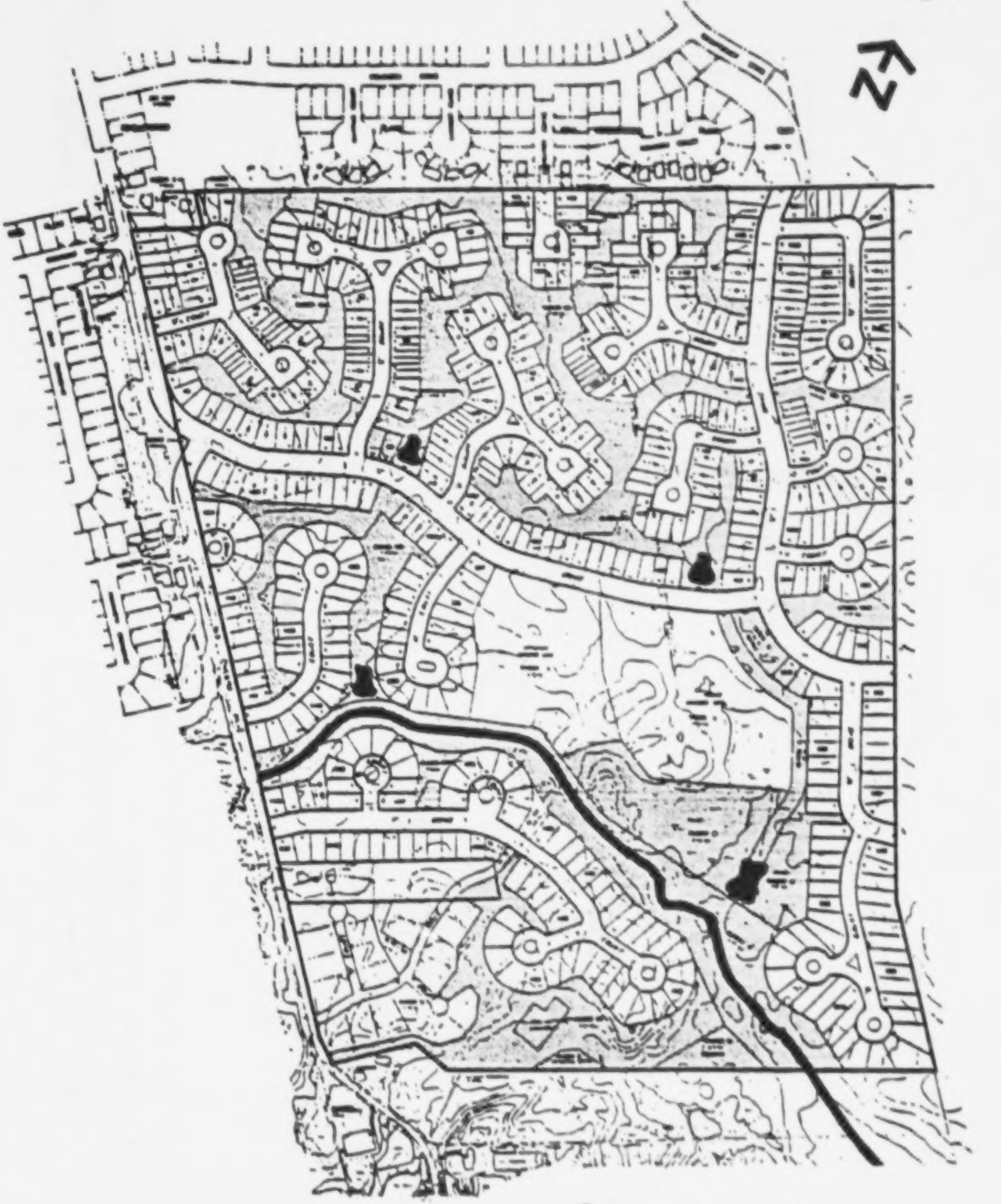

N
Proposed Storage Area

Figure 3 - Idlewood Subdivision Drainage Features

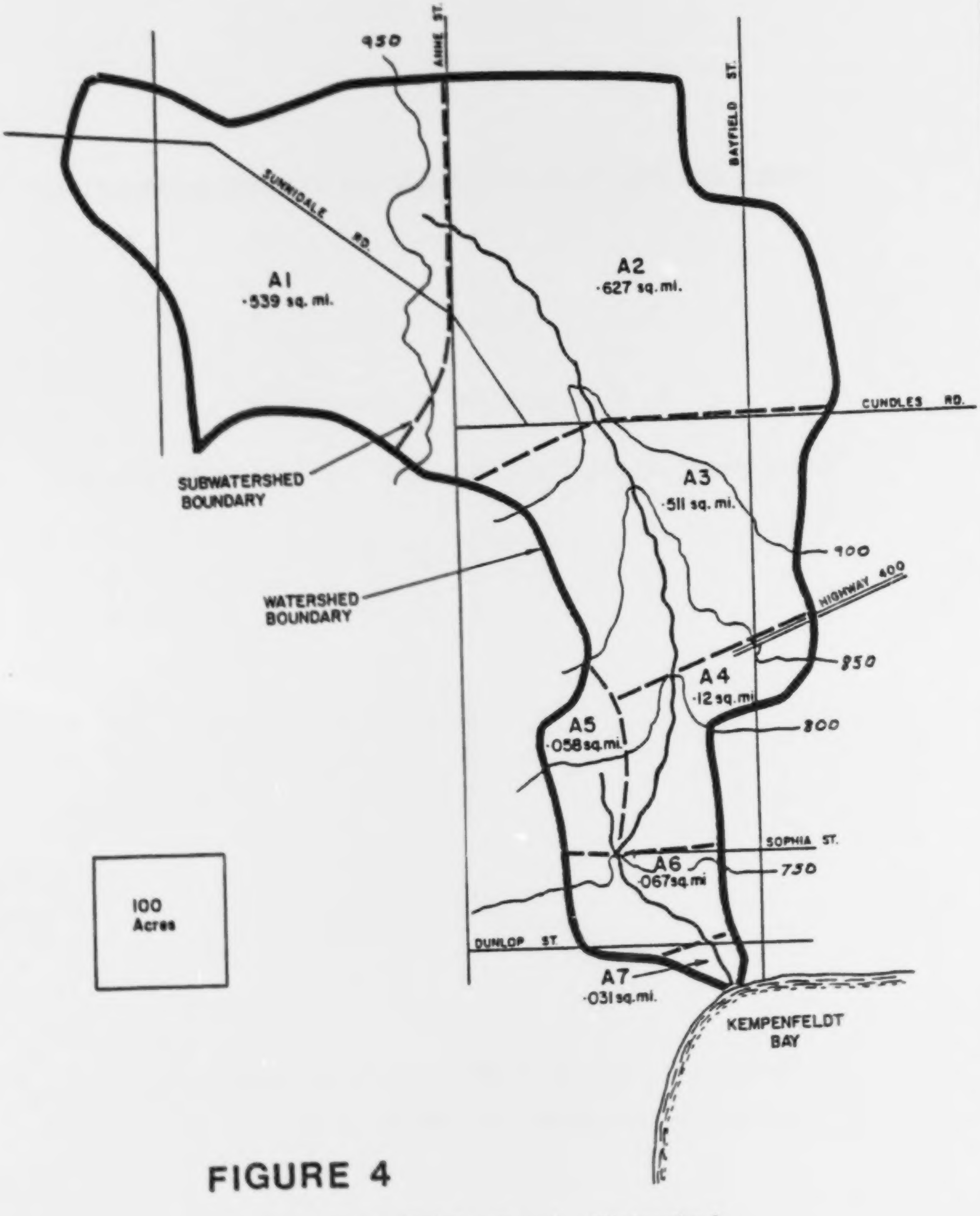

950
ANNE ST.
BAYFIELD ST.
SUNNIDALE RD.
A1
·539 sq. mi.
A2
·627 sq. mi.
CUNDLES RD.
SUBWATERSHED BOUNDARY
A3
·511 sq. mi.
900
WATERSHED BOUNDARY
HIGHWAY 400
850
A4
·12 sq. mi
800
A5
·058 sq. mi.
SOPHIA ST.
A6
·067 sq. mi
730
100 Acres
DUNLOP ST.
A7
·031 sq. mi.
KEMPENFELDT BAY

FIGURE 4

Schematic Of Kidd's Creek Watershed

"MODEL FOR SELECTION OF STORMWATER CONTROL ALTERNATIVES" *

by

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Department of Civil Engineering, University of Toronto

Toronto, Canada

* Paper presented at SWMM Users Group Meeting, Annapolis, Maryland, November, 1978

"MODEL FOR SELECTION OF STORMWATER CONTROL ALTERNATIVES"*

by

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ABSTRACT

In urban centres, the degree of pollution from combined sewer overflows and stormwater runoff is becoming ever more noticeable and critical. Among the many stormwater management alternatives for combined sewer overflow abatement, remote monitoring/real-time control is emerging as a cost-effective method in large urban centres. Its advantage is that it does not require additional facilities, but uses the existing system more effectively.

A general computer model is needed in the planning stage, which can estimate the overall performance of real-time control and other stormwater management alternatives, without employing expensive, single-event models like WREM. STORM, the most well-established initial planning model is unable to provide an overall system description. Therefore, a new program, named RAFFI, was devised which can be combined with STORM. RAFFI is a simple model which provides hourly information on flows and pollutants (BOD and SS) diverted into receiving waters from various overflow

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points, and treatment and by-pass operations at the plant.

The model can simulate the overall behaviour of existing and proposed systems in multi-basin urban areas for single storms, or for many storms over a long period, such as one year. In-system storage can be simulated by assuming that the interceptor acts as a common reservoir which can store flows from all watersheds, while each trunk can store flows only from the watershed that it serves. Dynamic regulators can be simulated according to a built-in control strategy, which changes the flow capacity of diversion pipes and outfalls, based on plant, trunk and interceptor flow and storage values at each hour. Capacities for primary, secondary or by-pass operations at the plant can be varied during storms. Treatment of overflows at each outfall can also be simulated. The model is equally applicable to separate storm sewer systems.

The objective was not to produce yet another new model, but to integrate program RAFFI with STORM, in order to increase its capabilities in the initial planning stage. By using STORM-RAFFI, a decision-maker can estimate the overall performance of an existing system during wet weather (i.e. quantity and quality of discharges from multiple overflow points, effluents from primary or secondary treatment, or by-passes at the plant), to compare the mass pollution loading from overflows with that from the plant, and to predict the long-term effect of alternatives such as real-time control with in-system storage, or overflow treatment. Selection of alternatives can be based on frequency, duration, volume and pollution of overflows, and on the total pollution (from all plant effluents plus overflows) reaching receiving waters during storms, all of which can be output from model RAFFI.

After the model was built and tested successfully on a hypothetical combined sewered area, it was applied to data from the City of Winnipeg, Manitoba,which experiences combined sewer overflow and plant by-pass problems. Results for a six-month simulation period indicated that the expansion underway at the plant will not reduce overflow pollution, even though it may eliminate by-passes.

Among the various abatement alternatives which were analyzed, real-time control with in-system storage distributed in the interceptor and trunk sewers produced the best results. Reductions were 20% for overflow frequency, 30% for overflow duration, 22% for overflow volume and 40-50% for overflow BOD and SS discharge, compared with the no-control system behaviour. Microstraining treatment of overflows at each point produced a 10-40% improvement in overflow quality but no change in overflow volume, frequency and duration. In comparison, all of the real-time control strategies, such as in-system storage in trunks only, or in line storage in the interceptor with holding reservoirs along its length, gave 20-50% improvements.

INTRODUCTION

Combined sewer overflows, stormwater runoff and treatment plant bypasses during storms are serious pollution sources in large urban centres. Until recently, the effects of runoff quantity on communities downstream (flooding due to increased flow), and of runoff quality on receiving water bodies (degradation due to shock loads) were underestimated, ignored or simply could not be determined. The prime objective of stormwater management was to collect runoff from the streets as fast as possible and discharge it to receiving waters. As a result of economical and environmental constraints, and increased monitoring of wet weather flow (WWF), the situation has changed. On the other hand, advances in treatment plant operations have improved the effluent quality significantly, thereby increasing the percentage contribution of combined sewer overflow/stormwater runoff in the overall pollution picture. Therefore, research has been directed towards better stormwater management. This paper is an attempt to provide a simple tool which may become quite useful in the planning phase of a stormwater management project.

PROBLEM

The urban water resources system is illustrated in Figure 1. The ultimate objective is to minimize the pollution discharge from paths 1 to 6 such that the downstream water quality will not be worse than the upstream quality. This paper deals with paths 3, 4, and 5 and provides a tool to investigate pollution abatement alternatives for them.

Combined sewer systems belong to the past; however, their pollution problems exist today and may even increase in the future. Increased dry weather flow (DWF), due to higher populations, will leave less capacity for additional flow to be diverted into treatment during storms; in parallel, increased imperviousness, due to urbanization, will create higher runoff, the net result being more frequent, longer and more polluted overflows.

Combined sewer overflows are more polluted in flat cities because low DWF velocities result in accumulation of pollutants in the sewer, only to be flushed during wet weather flow(WWF). Combined sewer overflwos, during the first hours in particular, create a shock load on receiving waters because they carry most of the accumulated pollutants picked up by precipitation from the air, by runoff from the ground surface, catchbasins and from within the sewer system, in addition to pollutants from mixing with municipal sewage. Usually, a runoff or overflow pollutograph reaches the peak much earlier than a runoff hydrograph causing high pollutant concentrations to be discharged at the start of a storm. Unlike the treatment plant effluent which is stable and has undergone biodegradation for several hours, the combined sewer overflow contains freshly produced pollutants and thus, has a high deoxygenation rate and can be easily degraded by all types of microorganisms present in receiving waters. Therefore, in the short term, an overflow with 30 mg/l BOD may have more detrimental effects than a plant effluent with 30 mg/l BOD, even though both will exert 30 mg/l BOD after 5 days. Various studies indicate that the annual sanitary sewage overflowed in combined sewers ranges from 2 to 5% (Field et al, 1973, Marsalek, 1972). Although this is a small volume, the pollutant contribution of this percentage is high. In an area with a secondary treatment plant, about 50% of SS and 30% of BOD reaching receiving waters annually come from overflows. Rosenkranz (1970) notes that 5% of the annual raw sewage load will become 33% of the effluent load, or 25% of the total load, if 85% reduction is achieved at the treatment plant. As the treatment methods improve and removal efficiencies increase, the pollution contribution of combined sewer overflows will become more critical.

SOLUTIONS

Stormwater management alternatives can be divided into four categories as shown in Table 1. Since references (Field et al, 1972, Lager et al, 1974, Marsalek, 1972) deal with all of them quite extensively, only the remote monitoring/real-time control method will be briefly explained.

Remote monitoring/real-time control is a collection system control that can be implemented in large cities with combined sewer systems. It has the objective of assisting a dispatcher (a human supervisor or control computer) in routing and storing combined sewer flows and making the most effective use of interceptor, trunk sewer and treatment plant capacities. This operation is controlled by predicting and monitoring precipitation, runoff, sewer flows and receiving water quality, and by accordingly regulating and distributing the WWF in the system before allowing any overflows. Those portions of the sewer system capacity not used for flow transmission are used for flow storage. Flows are stored in the pipes and distributed from high runoff to low runoff areas by means of dynamic regulators, gates and cross-connections. Backwatering and surcharging are allowed. Treatment plant throughput is regulated to maximize the amount of WWF treated. Overflows are allowed when all of the in-system storage is utilized and only from the least critical points. Information obtained from telemetered rain gauges, flow and depth sensors, quality monitors at outfalls, treatment plant and receiving waters are processed by a central computer, which calculate and implements the control operations, either automatically, based on pre-programmed strategies, or by the decisions of a human supervisor. Figure 2 shows a typical real-time control system.

Minneapolis-St. Pual, Minnesota (Anderson, 1970), Detroit, Michigan (Detroit Metro Water Department, 1970), and Seattle, Washington (Leiser, 1974) have applied this extensive but cost-effective concept and achieved substantial success in pollution reduction. Based on pre- and post-project conditions, average reductions are 35-70% in overflow volume, 68% in pollution, 68% in frequency and 88% in duration. Reported costs are $3-8 million capital and $200,000 - $270,000/year operational (Anderson, 1970, Leiser, 1974, Lager et al, 1974). This alternative is feasible and readily applicable in most large urban centres with combined sewer systems, provided that static regulators be replaced with dynamic ones.

TOOLS FOR SELECTING SOLUTION

Since substantial amounts of time and money are involved in each management alternative, thorough studies are required to evaluate each one. Computer models are tools needed by the decision-maker, designer and the operator of stormwater management facilities for selecting, analyzing, designing and implementing the most cost-effective method.

According to their objectives, the models can be classified into three groups:

1. Initial planning, 2. Detailed analysis/design, 3. Operation.

Data requirements, computer costs, level of sophistication and degree of accuracy increase, while output information, number of alternatives, length of study period and size of study area decrease from initial planning to detailed analysis and design models. In general, critical precipitation/runoff events can be identified and long-term effects can be estimated by the initial planning models by continuous simulation, while the single-event analysis/design models evaluate the short-term performance of existing or proposed control systems. Extensive information about current models and application results is provided by Brandstetter (1975), MacLaren and Procter Redfern (1976), Water Resources Engineers et al (1974).

NEED FOR A NEW TOOL

At present, there is no initial planning model to study stormwater management alternatives such as real-time control or stormwater overflow treatment, although models do exist to analyze and design them in the latter part of a management project. At the start of a project, it is a waste of time and money to use a highly sophisticated model which requires much input data and outputs much detailed and unneeded information about each alternative. In addition, models in the design/analysis class are single-event simulation models and therefore, cannot estimate the overall, long-term behaviour of a control alternative (or of the existing system), which is essential for decision-making and economic analysis.

Real-time control is becoming an attractive abatement alternative in large combined sewered cities. After some success in a few American cities, some Canadian municipalities, Montreal, Toronto and Edmonton have also begun to consider this concept. The Metro Toronto sewer system is similar to that of Minneapolis-St. Paul, being equipped with dynamic gates and water level sensors which are adaptable to real-time control operations. Although the hardware exists, the software to estimate and analyze effective ways of real-time control do not exist yet. Decision-makers will require some general answers to the following questions before they consider any further action in this direction: What will happen with real-time control? Is it worth the effort, time and money? How much reduction in overflow frequency, volume and pollution will be achieved with real-time control? Will overflow treatment be any better? etc.

An initial planning study with the model proposed in this paper could answer some of these questions. It would, at least, provide a "feeling" of the system and possible solutions. This type of study would not benefit from sophisticated models like Water Resources Engineers Model (WREM), because of high cost, high degree of detail in input data, unneeded and detailed information in output and shortness of test period (single events).

Storm Treatment Overflow Runoff Model (STORM), the most established initial planning model (U.S. Army Corps, 1975), is not suitable in this area because it can only analyze quantity and quality of one outfall from one watershed at a time, whereas a total system contains many watersheds with many outfalls and, in case of a combined system, a treatment plant whose performance changes during storms. Futhermore, STORM cannot simulate variable treatment and variable storage during storms, which are reqired operations for a real-time control alternative.

OBJECTIVES OF RAFFI

In order to overcome the deficiencies of STORM, a new model named RAFFI was built. Its objectives are:

1. To investigate the overall effects of a storm, or of many storms in a given period, in a multi basin urban area; more precisely, to provide hourly information on flows and pollutants (BOD and SS) reaching receiving waters from multiple overflow points, treatment plant secondary and primary effluents, or by-passes; to analyze percentage breakdown of all WWF quantity and quality, as they reach receiving waters from various components of the system.
2. To analyze any improvements due to real-time control in combined sewers - by variable in-system storage in interceptor and trunks, and variable treatment at the treatment plant.
3. To analyze any improvements due to combined sewer overflow or separate stormwater treatment at outfalls before discharging to receiving waters.
4. To compare pollution discharged from overflows/stormwater runoff, with that from treatment plant effluents or by-passes.
5. Along with another program built at the University of Toronto, to increase capabilities of STORM to analyze most of the available stormwater management alternatives.

In addition to the above, the model computes information on the number of events contained in the system without overflowing, and the average duration of overflows at each point. The output can be (i) a summary for all events in a continuous simulation; (ii) a summary for a single-event, and (iii) a detailed hourly analysis of an event for overflow hydrograph and pollutograph construction. It contains about 500 statements and needs an average of 5 seconds to analyze a system with 12 watersheds for a six-month continuous simulation. The model assumptions, flowchart, program listing, applications and results are described in detail elsewhere (Bedrosyan, 1976).

Figure 3 shows an overview of model RAFFI. It can analyze up to 20 watersheds simultaneously, which may have separate storm sewers, or may be connected to a treatment plant with a combined sewer system. Input to the model is in the form of hourly runoff hydrographs and BOD and SS pollutographs for each watershed. STORM output can be used as RAFFI input, although the input data may be supplied manually or from any other model that can generate runoff from rainfall. Computations are in hourly time-steps; units are acre-inch/hr for flow routing, lb/hr for pollutant routing and acre-inch for storage routing. The intention of this work is not to produce yet another new model but to integrate it with STORM, in order to increase its capabilities in the initial planning stage.

ASSUMPTIONS

The important assumptions used in formulating the model are briefly discussed below:

1. Each watershed is served by one major trunk sewer. In combined sewer systems, a diversion pipe connects the trunk to the interceptor, which collects flows up to a low multiple of DWF from each watershed and transports them to the treatment plant. Flows in excess of the diversion pipe or regulator capacity are overflowed from each trunk. DWF from each

watershed can be input as average flow in cfs and BOD and SS concentrations in mg/l.

2. Each trunk-interceptor connection is equipped with a dynamic regulator, which can change the diverted flow, according to plant treatment rates or available storage space in the interceptor. The range of regulation is from zero (regulator gate closed) to diversion pipe capacity (gate fully open). Conventional combined systems are simulated by assuming inactive regulators, set at a fixed diversion capacity above which all excess flows in a trunk (i.e. flows above 2.5 x DWF) are overflowed.

3. In-system storage can be simulated in any trunk or interceptor by assuming a pipe to act like a reservoir. The lower limit of storage is zero (i.e. if the pipe is flowing full) while the upper limit is input by the user. It can be computed as follows:

Maximum storage available = (free cross section) x (safe length),

where (free cross section) = cross section of pipe less area used by flow,

(safe length) = a distance upstream beyond which basement flooding or high head may be critical.

4. The treatment plant is simulated by two variable treatment operations with two outflows. The outflows may be chosen by the user, such as (i) secondary effluent and (ii) primary effluent, or (i) treated effluent, and (ii) untreated by-pass, etc. The treatment capacities of the two operations are input in multiples of average total DWF and percent BOD and SS removal efficiencies. It is assumed that an increase in flow during storms will not affect the performance of a large plant, if

this increase lies within certain hydraulic and/or organic limits and is not prolonged. Variations in inflow are much more critical for a small plant than for a large one; however, most of the combined sewered areas are old and highly populated urban centres, served by large treatment plants with average DWF's greater than 50 Imperial million gallons per day (Imgd).

The secondary (biological) treatment process, designed for average DWF, is more susceptible to inflow variations than the primary operations. Based on plant performance data (Heinke, 1973, Lin, 1974, Salib, 1976), it is generalized that, for a plant larger than 50 Imgd DWF, flow increases up to 1.5 - 2.0 x DWF will not affect the biomass activity, if this increase does not last longer than 6-15 hours. The momentum of the process is stable enough to allow increases within these limits. BOD and SS removal efficiencies may be in the range of 70 - 95%.

The primary (physical-chemical) treatment operations, designed for maximum day DWF (usually 2-3 x average DWF), are readily available for WWF treatment if DWF is at its average during the storm. Additionally, most settling tanks have allowances for up to 1.5 - 2.0 x the design surface loading rates (Heinke, 1973, Salib, 1976). Removal efficiencies will decrease slowly with increasing surface loading rates until a certain limit is reached, beyond which there will be a sharp decline in efficiencies and a wash out of pollutants (Heinke, 1973). It is generalized that flow increases up to 2.0 - 4.0 x DWF will not affect primary operations and that BOD and SS removal efficiencies may be 30 and 50%, respectively. In the developed model, the capacity limits and removal efficiencies are not defaulted but left to the user's choice, for greater flexibility.

5. Overflow treatment at each outfall can be simulated and different treatment methods for different overflow points can be studied. Physical or physical-chemical operations such as microstraining, filtration, screening/dissolved-air flotation can be simulated. Most models use steady-state operations to simulate stormwater treatment, even though stormwater is a dynamic and highly variable phenomenon; these formulae give results with a wide range of fluctuations (Lager, et al, 1974, MacLaren-Proctor Redfern, 1976, Wisner, 1976) yet the mean values are calibrated to match actual results. Since equations are not any more accurate, and since the objective in initial planning is to estimate the long-term behaviour of a treatment alternative, the only input data needed is percent BOD and SS average removal efficiencies, and constraints of minimum feasible effluent concentrations, which can be directly obtained from some pilot plant studies (Glover et al, 1973, Gupta et al, 1973, Harvey et al, 1973, Lager et al, 1974, Lee et al, 1974, White et al, 1973). There is a need for specifying minimum effluent concentrations during simulation because stormwater runoff/combined sewer overflows, unlike municipal sewage, are highly variable, both in flowrate and pollutant concentration; a treatment unit cannot be equally effective across the wide range of inflow quantity and quality. Effluent from a microstraining unit, for example, cannot have less than 10 mg/l SS, due to physical limitations such as screen size, flux, etc. If the average SS removal efficiency is 80% and the influent concentration 20 mg/l, the effluent cannot be 20 (1.0 - 0.80) = 4 mg/l, but 10 mg/l at the lowest. Therefore, the program assigns the minimum feasible effluent concentrations when the effluent or the influent is below these limits.

6. Real-time control of in-system variable storage and variable treatment at the plant is simulated according to a built-in strategy. This strategy is adopted in accordance with physical constraints and the results of various strategies reported in literature (Anderson 1970, Detroit Metro Water Department, 1970, Bell, 1974, Leiser, 1974, Wenzel et al, 1974). It is assumed that the interceptor acts as a common reservoir and can store flows from all watersheds, while a trunk can store flows only from the watershed that it serves. The main advantage of interceptor storage over trunk storage is that the interceptor is long enough to distribute flows during moving storms, from rainy areas of the city to the dry areas, thereby preventing or at least retarding overflows.

When a storm begins,and before any in-system storage is utilized,the treatment rates at the plant are increased to their maximum capacities, in order to capture the polluted "first flush" at the beginning of the storm. If the storm intensities generate runoff greater than the treatment rates, the sewers are used for storage. Priority is given to interceptor storage because the interceptor is long enough to distribute flows from high runoff areas to low runoff areas of the city, during localized storms. Another reason for interceptor storage to precede trunk storage is that filling the individual trunks first may cause overflows before there is a chance to fill the interceptor. This may happen if the runoff intensity exceeds the reduced flow capacity of the trunk, or the flow capacity of the regulator/diversion pipe. In this case, overflowing will be essential in order to prevent flooding, even though interceptor storage capacity still exists. Therefore, once the runoff intensity exceeds the maximum treatment rates, interceptor storage is utilized; if the storm still continues after the interceptor is filled up, the trunks are used for storage; and if these are also filled, only then are overflows allowed. The recession of the storm and emptying of the system follows a similar pattern; the treatment rates are decreased to average DWF only when all of the in-system storage is emptied and treated. The control strategy is described in Figure 4,and the performance of each system component shown in Figure 5. Routing is computed, based on mass balance procedures, in hourly time-steps with continuous updating of plant, trunk, interceptor flow and storage values. Pollutant movement along the system is assumed to be flow-proportional.

7. The model is equally applicable for separate storm system simulation. By inputting zero trunk-to-interceptor diversion pipe capacity, the combined system is converted to a separate one. Various stormwater treatment methods can be simulated continuously with multiple watersheds and outfall points.

APPLICATION

The model was built and tested on a hypothetical combined sewered city. Performance of a conventional combined system and various real-time control alternatives were tested both for single events, and for all events in a one-year period. Results similar to those reported in Seattle (Leiser, 1974) and Detroit (Detroit Metro Water Dept., 1970) were obtained.

Next, the model was applied to one of the three interceptor/treatment plant systems in the City of Winnipeg, Manitoba, which experiences combined sewer overflow and plant by-pass problems. The study area shown in Figure 6, is about 9,300 acres, generates about 15 Imgd DWF and is serviced by a 10-mile long interceptor which lies along the Assiniboine and Red Rivers. There are twelve watersheds, each served by a combined trunk sewer that can divert up to 2.75 x DWF into the interceptor/plant system, all excesses being overflowed. The plant at the end of the interceptor is the North-End Sewage Treatment Plant, which is quite overloaded; the average DWF that it treats is near its design capacity. This situation has resulted in frequent by-passes during WWF, sometimes even during DWF. Therefore, the City is enlarging the plant, from 55 to 100 Imgd for secondary and from 100 to 180 Imgd for primary treatment capacities (City of Winnipeg, 1975, MacLaren, 1976).

Precipitation records from three different rain gauges were used to illustrate the spatial distribution and movement of the storms. STORM was applied to each of the twelve watershed to produce runoff hydrographs and BOD, SS pollutographs for all events in a six-month period. Then, RAFFI was used to combine the output of STORM and to simulate the overall behaviour of the existing system and the system with the proposed control alternatives. DWF was simulated by inputting average flow and pollutant data for each watershed.

Apart from the existing system simulations, three real-time control, and two overflow treatment alternatives, and all their combinations were analyzed as seen in Table 2.

The existing or no-control alternative was assumed to be the Winnipeg system after the expansion underway at the plant was completed. The simulations showed that this plant expansion will have virtually no effect on the overflow pollution, even though it may eliminate the bypasses.

In the simulation of the existing system, it was found that total runoff duration was 330 hours, 4% of the 6-month simulation period; yet, the BOD and SS loads discharged to the receiving waters during WWF (4% of the time) were equal to, or greater than the loads discharged during DWF (96% of the time), based on the assumption of 80 - 90% removal efficiency of the plant. This finding clearly indicated that the WWF pollution was quite significant; moreover, it took place in a short period of time and thus, its short-term effects were even more critical.

Results of the continuous simulation are shown in Figure 7. It can be seen that in the expanded system, without any control,nearly 80% of the total WWF volume and 30 - 40% of the total WWF pollution (or 60 - 80% of the total pollution discharged out) reached receiving waters from overflows, during all storms in a six-month period. Overflow frequency was 100% (all events overflowed), and average overflow duration was 3.6 hours per event. Among the various control alternatives, B3, real-time control with in-system storage distributed in trunks and interceptor, produced the best results. Improvements were 20% for overflow frequency, 30% for overflow duration, 22% for overflow volume and 40 - 50% for overflow BOD and SS discharge, compared with the no-control system behaviour. By increasing the treatment rate and by storing the WWF in various parts of the system, overflows during the first hours of an event were prevented, thereby protecting the receiving waters from the polluted first flush.

Treatment of overflows was not as effective as real-time control because the minimum feasible effluent concentrations were usually higher than the effluent concentrations to be achieved by the removal efficiencies; therefore, the facilities could not be used at optimum efficiency. Improvements ranged in 10 - 40% in overflow quality, without any change in overflow volume, duration and frequency. In comparison, all of the real-time control strategies produced 20 - 50% improvements, based on results for the no-control alternative.

Short-term performance of the alternatives was also analyzed. Simulations with test storms and selected events showed that real-time control alternatives could keep the WWF in the system for the first 1.0 - 3.0 hours of an average 6-hour storm, before allowing overflows. Results were much more favourable during moving storms; this clearly showed the advantage of "distributable" storage in the system for effective capacity utilization. Figures 8 and 9 are examples of the results obtained from the detailed analysis of a 6-hour event.

LIMITATIONS

Once the model is built and its potential use demonstrated, its capabilities may be increased and limitations reduced, using this work as a base or starting point. Some of the limitations and recommendations for improvement are mentioned below:

1. The model uses only one real-time control strategy, i.e. one method of sequencing the operations. Work should be done to provide more than one control strategy and regulator operation rule.

2. Since every city has its own unique sewer network configuration, the model should be able to handle more than one type of sewer system.

Analysis of two or more interceptors (as in Toronto), two or more overflow and regulator points in a watershed should be incorporated; but the program is flexible and most users can handle these changes.

3. Time lag during routing is not provided.

4. Quality change during system control is ignored, even though in reality, there will be settling of pollutants in the sewers if stormwater is retained in the pipes for 6-8 hours. Quality routing is assumed to be flow-proportional.

5. Performance of treatment plant and overflow treatment facilities are approximated as percent removal efficiencies, within some specified limits and constraints.

6. Interfacing RAFFI with STORM is one of the challenging limitations. However, this limitation is not in model logic but only in computer programming software and can be solved.

STORM simulates runoff events from a given precipitation record for one watershed at a time. Only after the entire record is simulated for one watershed, does it start on the next watershed and repeat the same procedure. RAFFI, on the other hand, simulates one runoff event from all watersheds and analyzes simultaneously the effects of this event on the total system. When the event has ended, it proceeds to the next event and covers all watersheds and the common system again. The two models can be visualized to operate as the row and column of a matrix. Memory storage or programmed punching of STORM output are not feasible solutions because of the size of data output and card sequencing. A new subroutine should be added to STORM, which will enable the model to proceed hourly and provide "absolute" events rather than "relative" ones; in this fashion, STORM should be able to identify the first hour of an event in absolute terms. That is, as soon as runoff begins in one watershed, it should register in all watersheds as a new event, regardless of whether there is runoff in other wathersheds or not.

7. The incorporation of an economic analysis and/or optimization model to the STORM-RAFFI package could be the next step in development which would increase the capabilities of the decision-maker. Optimization models for this purpose are discussed by Bell (1974), Lindholm (1974), Wenzel et al (1975).

CONCLUSIONS

This work essentially demonstrates two things - first, that real-time control is a cost-effective abatement alternative and second, that model RAFFI can be used to evaluate this in the initial planning stage. Despite the high initial cost, real-time control helps to utilize all components of an existing system very effectively. Since it can be implemented readily in most major combined sewered cities without requiring any additional facilities, it should be considered an attractive solution. Despite its limitations and simplistic assumptions, RAFFI can become a useful tool when interfaced with STORM, in the initial planning stage of a stormwater management study. It is suggested that at the beginning of a project, the behaviour of an existing system and all alternative abatement methods be estimated with a simple,inexpensive,continuous simulation model like STORM-RAFFI over a long period. This will provide a starting point and will help to screen a large number of alternatives at a low cost.

Once a feeling of the problem and potential solutions is obtained, the number of alternatives can be reduced and it may become feasible to concentrate on the more effective ones with more detailed and expensive models such as WREM (Water Resources Engineers Model) or DORSCH-HVM (Hydrograph Volume Method).

Finally, an example is cited which may help to indicate the potential for STORM-RAFFI. Simulation with WREM on the Winnipeg study area would cost \$300 - \$400/run, for a single 6-hour critical event, with surcharged conditions (Clarke, 1976). The output would be in the form of very detailed overflow hydrographs, water depths, height of surcharge, etc., at many "nodes" for time intervals of less than a minute. The study with STORM-RAFFI, would cost only \$15 - \$20/run, for a continuous six-month record, with much more useful information to the decision-maker about real-time control or overflow treatment alternatives.

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Table 1

Stormwater Management Alternatives

1. Sewer Separation
2. Source Controls
   - (i) on-site storage
   - (ii) improved street cleaning practice
   - (iii) wiser usage of chemicals
3. Collection system controls
   - (i) infiltration/inflow control
   - (ii) sewer flushing
   - (iii) polymer injection
   - (iv) remote monitoring/real-time control
4. Storage/Treatment
   - Storage
     - (i) Upstream - in-line or off-line
     - (ii) Downstrean - in-line or off-line
     - (iii) In-system
   - Treatment
     - (iv) Physical microstraining, filtration
     - (v) Physical-Chemical disinfection
     - (vi) Biological - biological contactor
   - (vii) Combinations of (i) to (vi)

TABLE 2

ALTERNATIVES ANALYZED

1. NO CONTROL
2. OVERFLOW TREATMENT
    (i) Low-rate
    (ii) High-rate
3. REAL-TIME CONTROL, IN-SYSTEM STORAGE
    (i) In trunks
    (ii) In trunks + interceptor
    (iii) In interceptor + common reservoirs
4. COMBINATIONS OF 1, 2, 3.

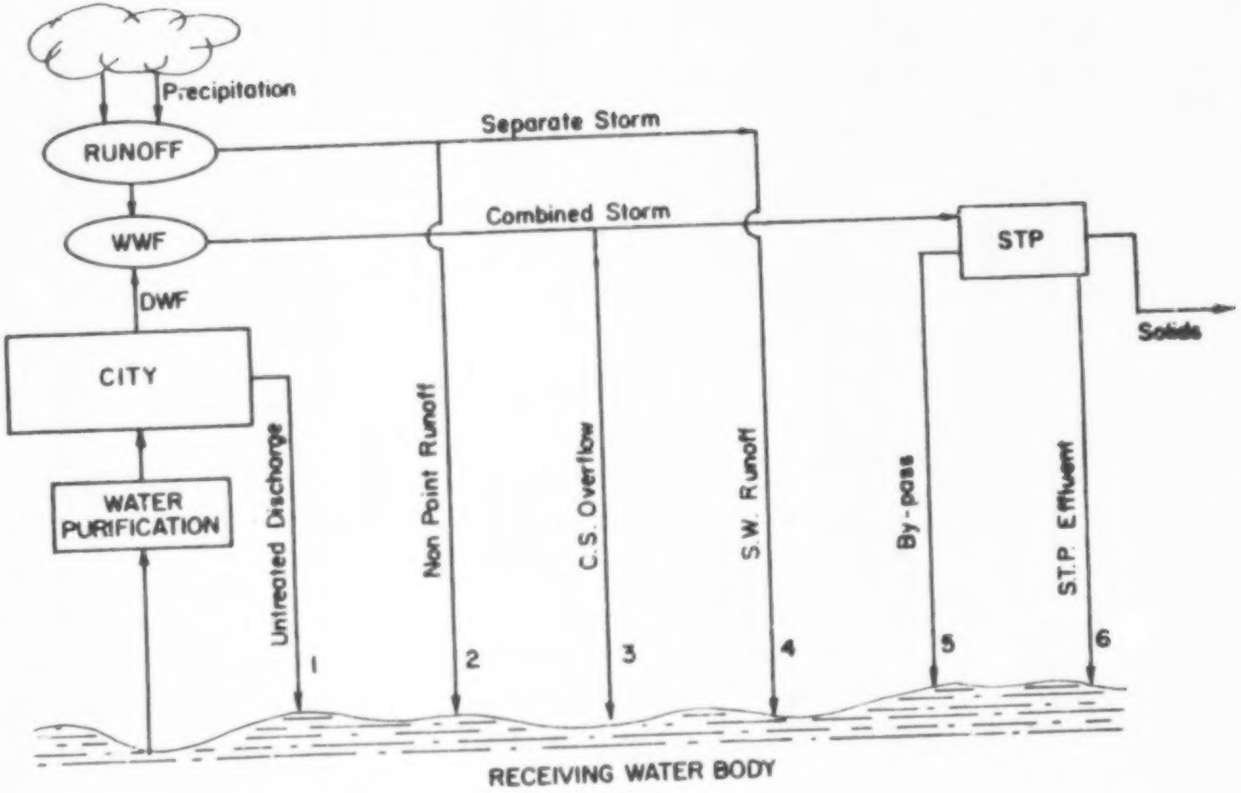

Precipitation
RUNOFF
WWF
DWF
CITY
WATER PURIFICATION
Separate Storm
Combined Storm
STP
Solids
Untreated Discharge
Non Point Runoff
C.S. Overflow
S.W. Runoff
By-pass
S.T.P. Effluent
1
2
3
4
5
6
RECEIVING WATER BODY

FIG. I WATER RESOURCES SYSTEM OF AN URBAN AREA

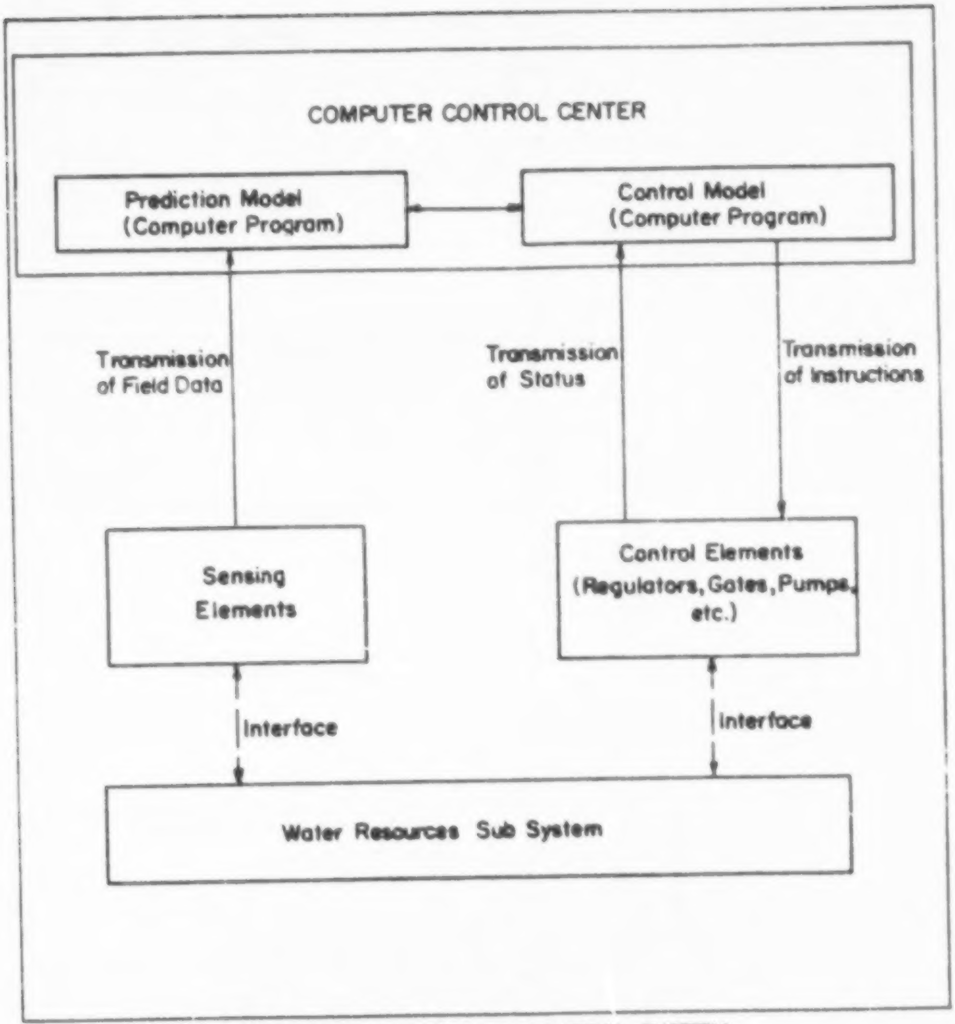

COMPUTER CONTROL CENTER
Prediction Model (Computer Program)
Control Model (Computer Program)
Transmission of Field Data
Transmission of Status
Transmission of Instructions
Sensing Elements
Control Elements (Regulators, Gates, Pumps, etc.)
Interface
Interface
Water Resources Sub System

FIG. 2 A COMBINED SEWER CONTROL SYSTEM

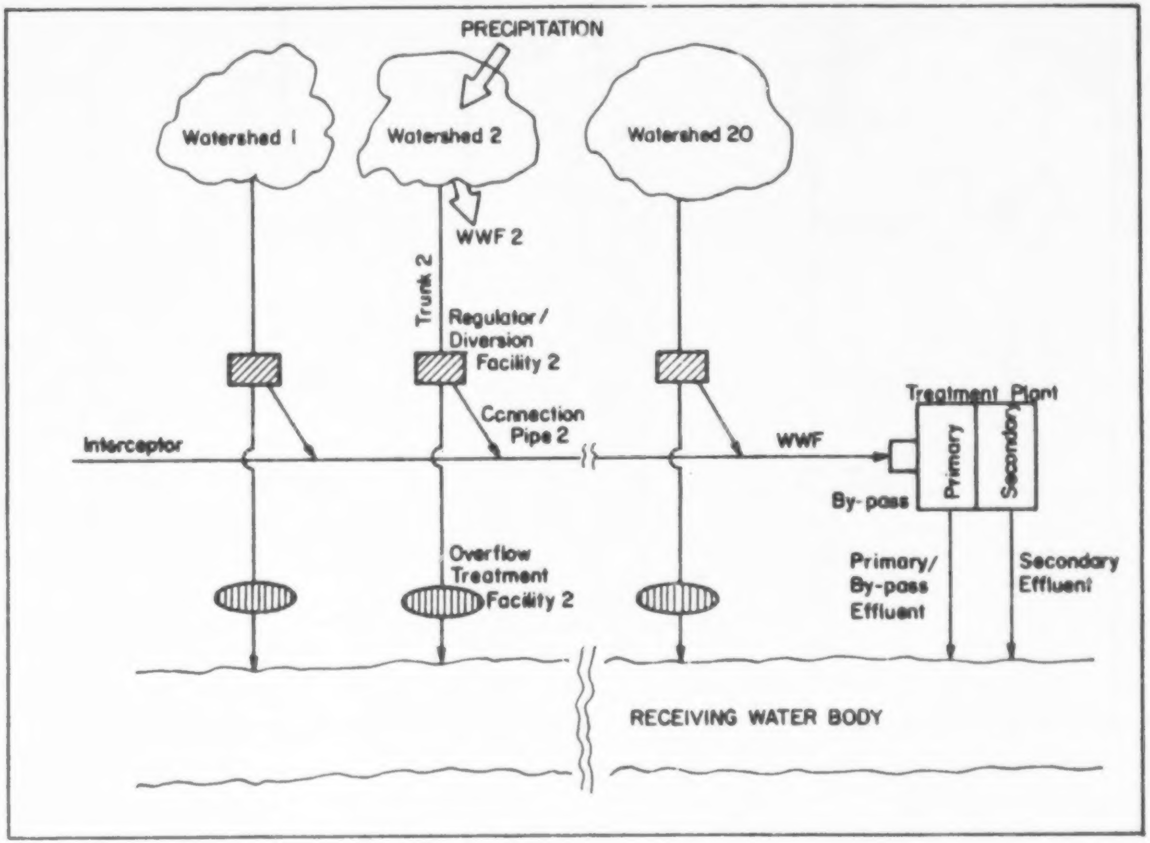
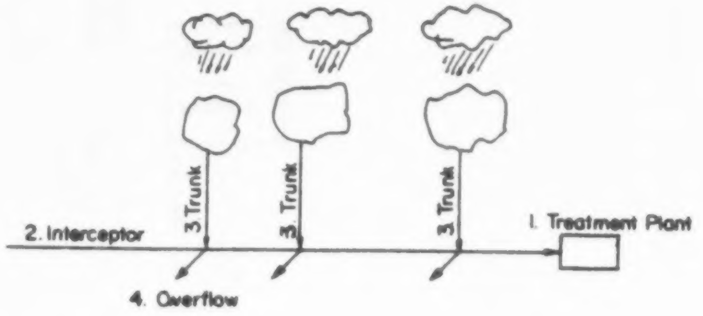

PRECIPITATION
Watershed 1
Watershed 2
Watershed 20
WWF 2
Trunk 2
Regulator/
Diversion
Facility 2
Treatment Plant
Connection
Pipe 2
Interceptor
WWF
Primary
Secondary
By-pass
Overflow
Treatment
Facility 2
Primary/
By-pass
Effluent
Secondary
Effluent
RECEIVING WATER BODY

FIG. 3 RAFFI- An Overview

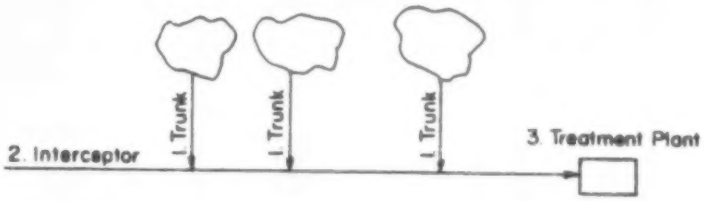

3 Trunk
3 Trunk
3 Trunk
2. Interceptor
1. Treatment Plant
4. Overflow

STRATEGY

1. Increase treatment plant capacity to maximum, T
2. Fill the common interceptor - subject to constraint of connection pipe capacities, and if R>T
3. Fill the individual trunks, if interceptor filled and R>T
4. Overflow, if R>Regulated inflow capacity

R - Runoff

(a) SYSTEM FILLING STRATEGY


1 Trunk
1 Trunk
1 Trunk
2. Interceptor
3. Treatment Plant

STRATEGY

1. Empty the trunks at T
2. Empty the interceptor at T
3. Reduce treatment rates from maximum, T, to normal DWF

(b) SYSTEM EMPTYING STRATEGY

FIG. 4 CONTROL STRATEGY IN REAL-TIME CONTROL

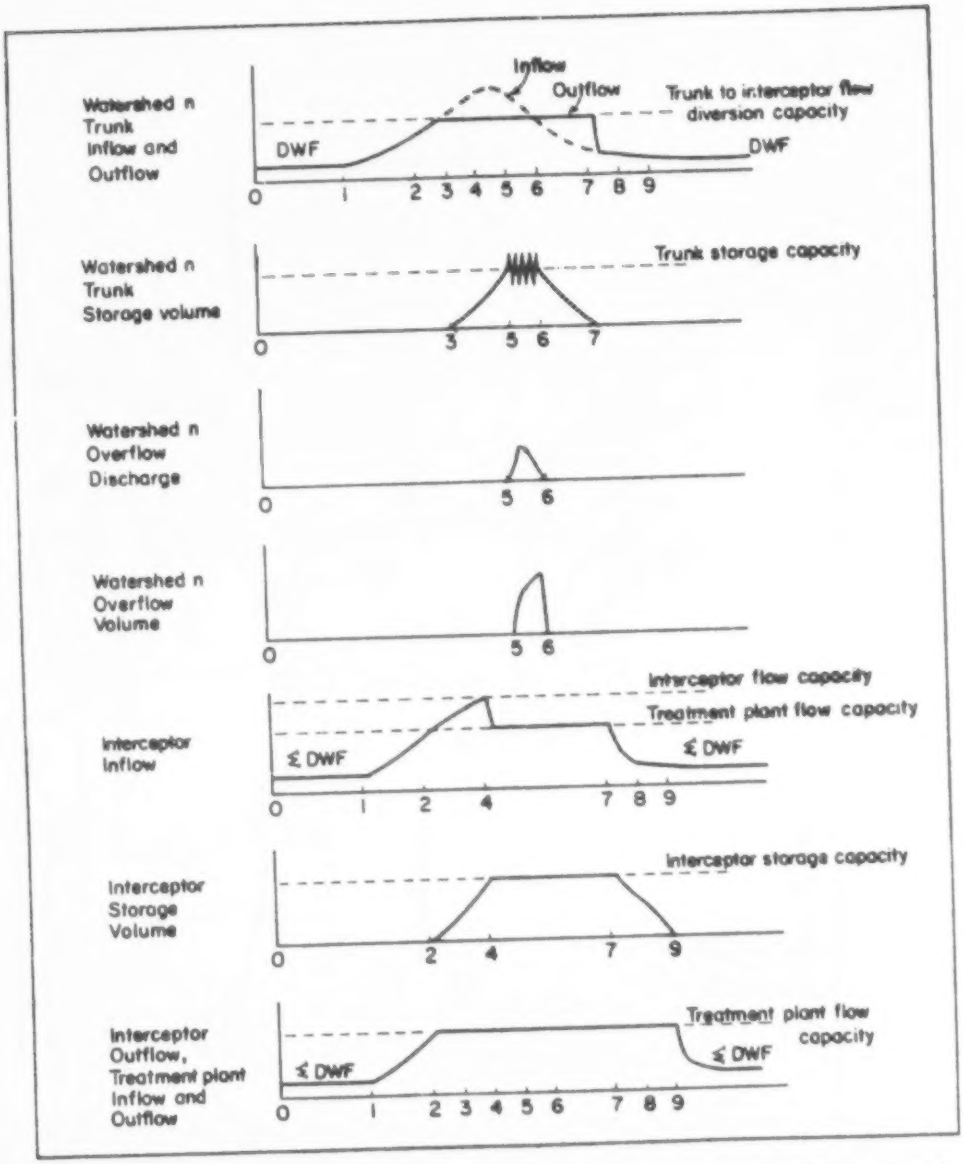


FIG. 5 PERFORMANCE OF SYSTEM COMPONENTS DURING REAL-TIME CONTROL

FIG. 6 CITY OF WINNIPEG

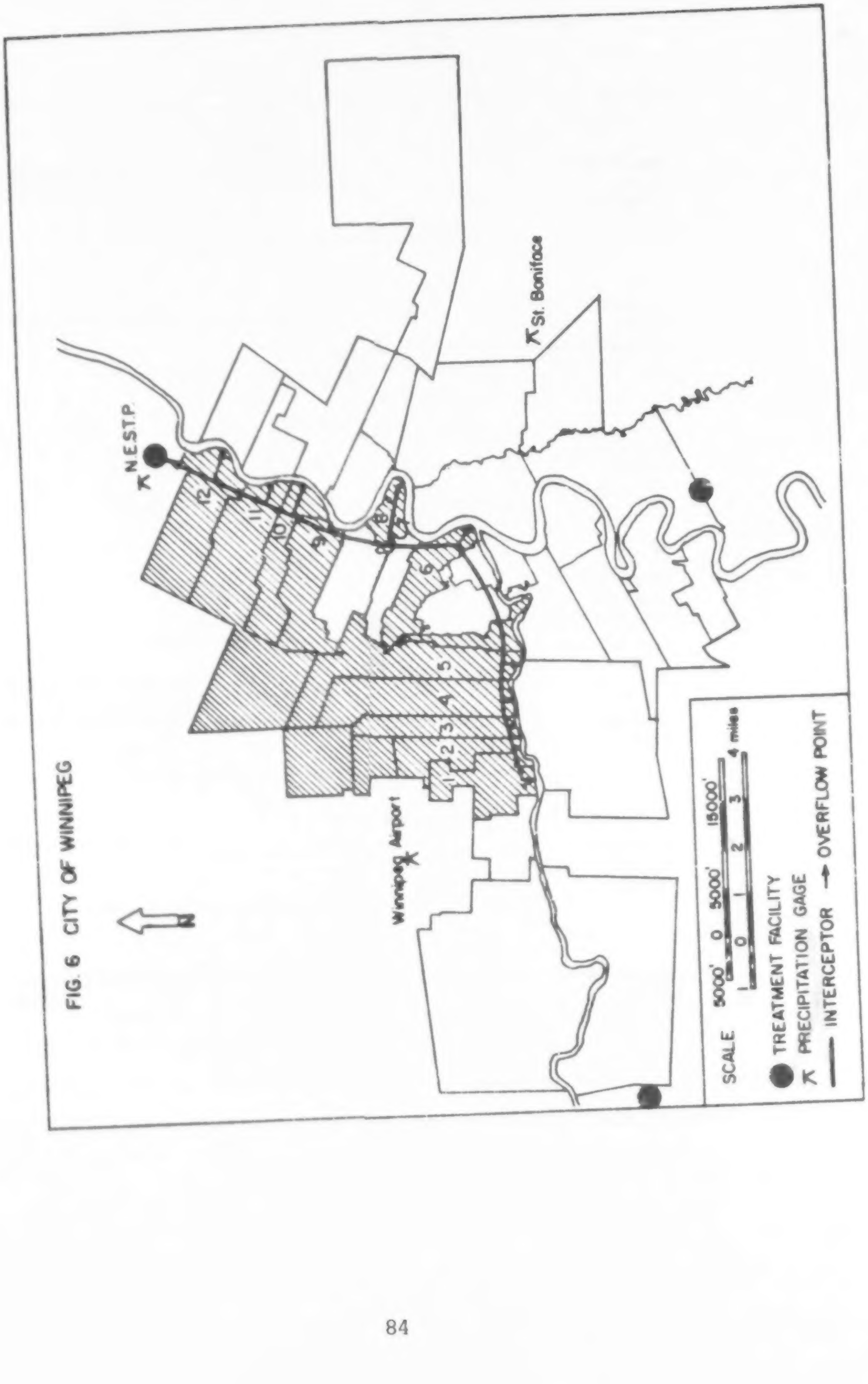
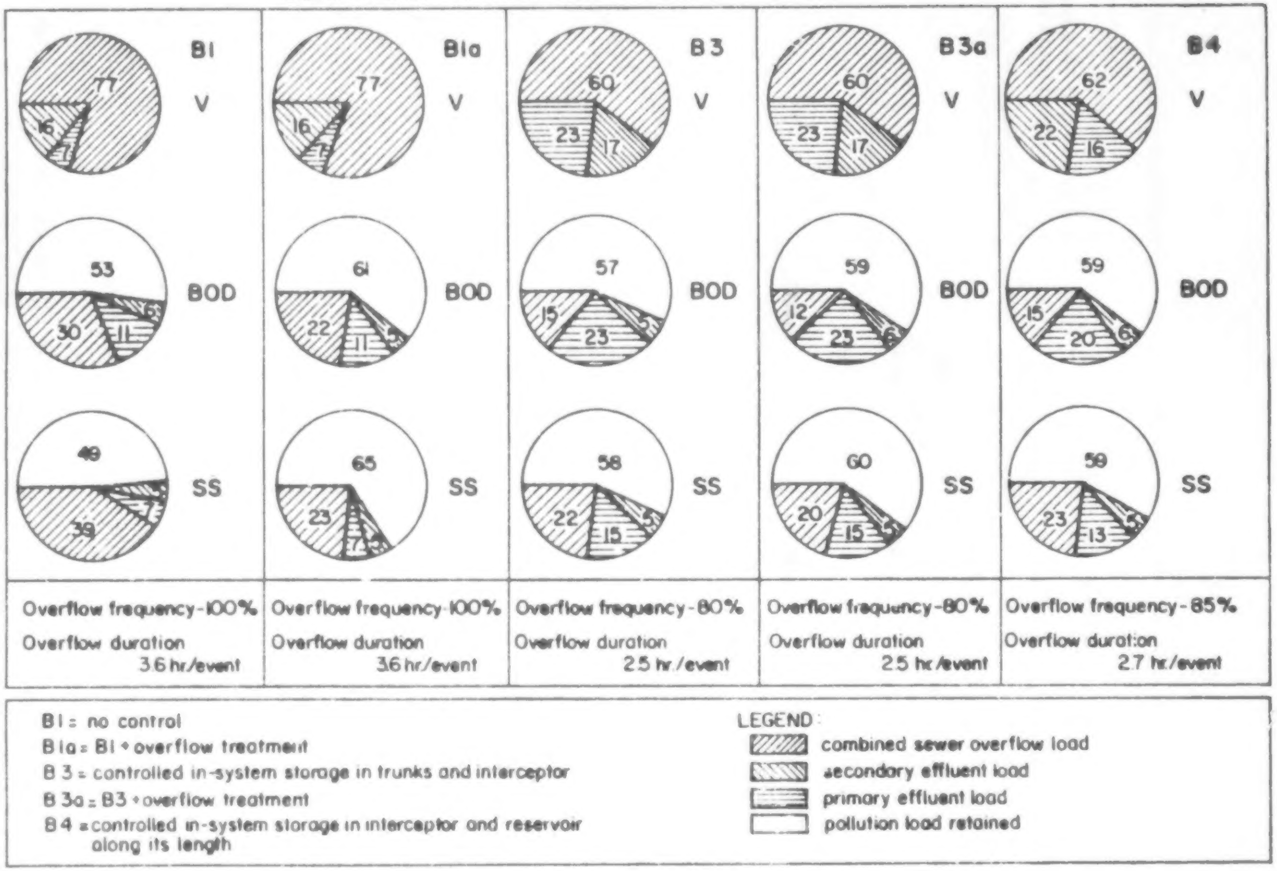

N
N.E.S.T.P.
St. Boniface
Winnipeg Airport
1
2
3
4
5
6
7
8
9
10
11
12
SCALE 5000' 0 5000' 15000'
1 0 1 2 3 4 miles
● TREATMENT FACILITY
⊼ PRECIPITATION GAGE
— INTERCEPTOR → OVERFLOW POINT

FIG. 7 PERCENTAGE POLLUTION LOADS REACHING RECEIVING WATERS DURING WWF IN A SIX-MONTH PERIOD - WINNIPEG



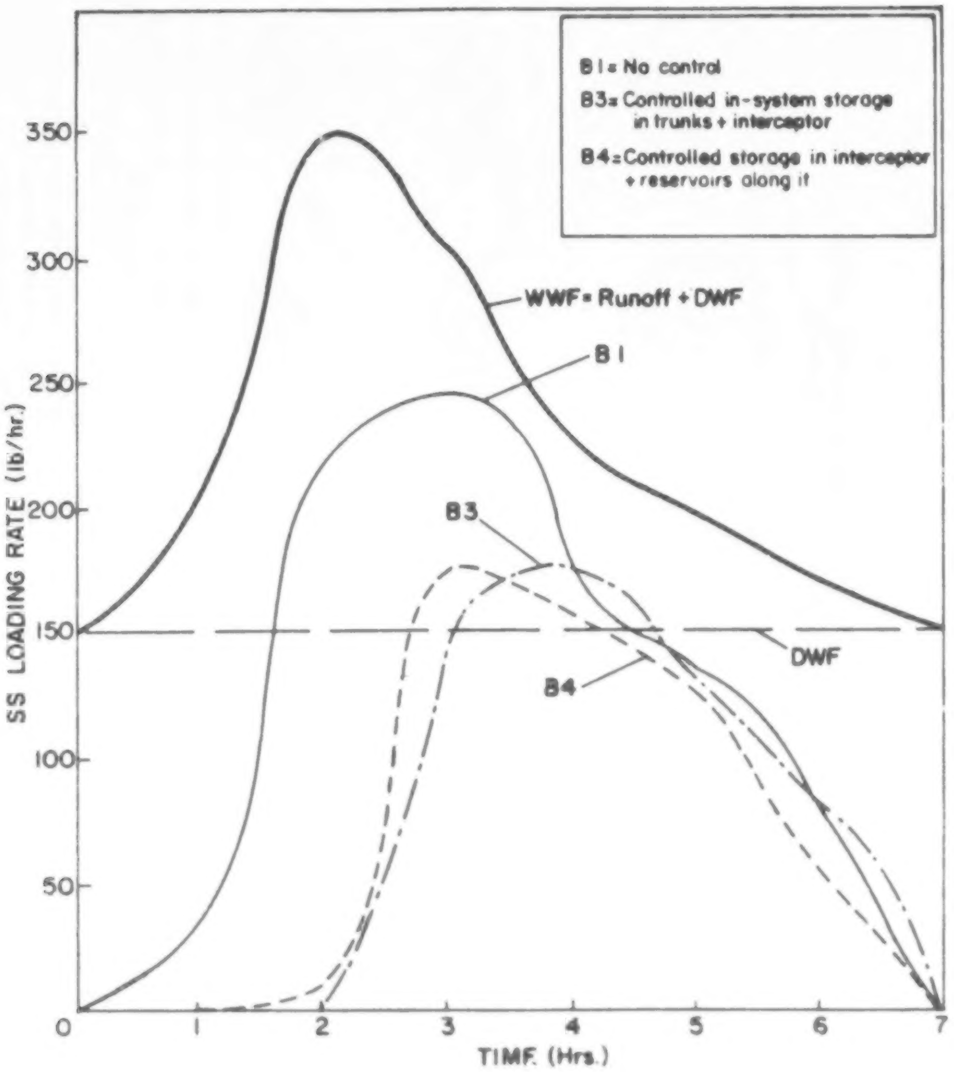


FIG. 8 WINNIPEG-OVERFLOW SS POLLUTOGRAPHS SINGLE EVENTS FOR SOME ALTERNATIVES

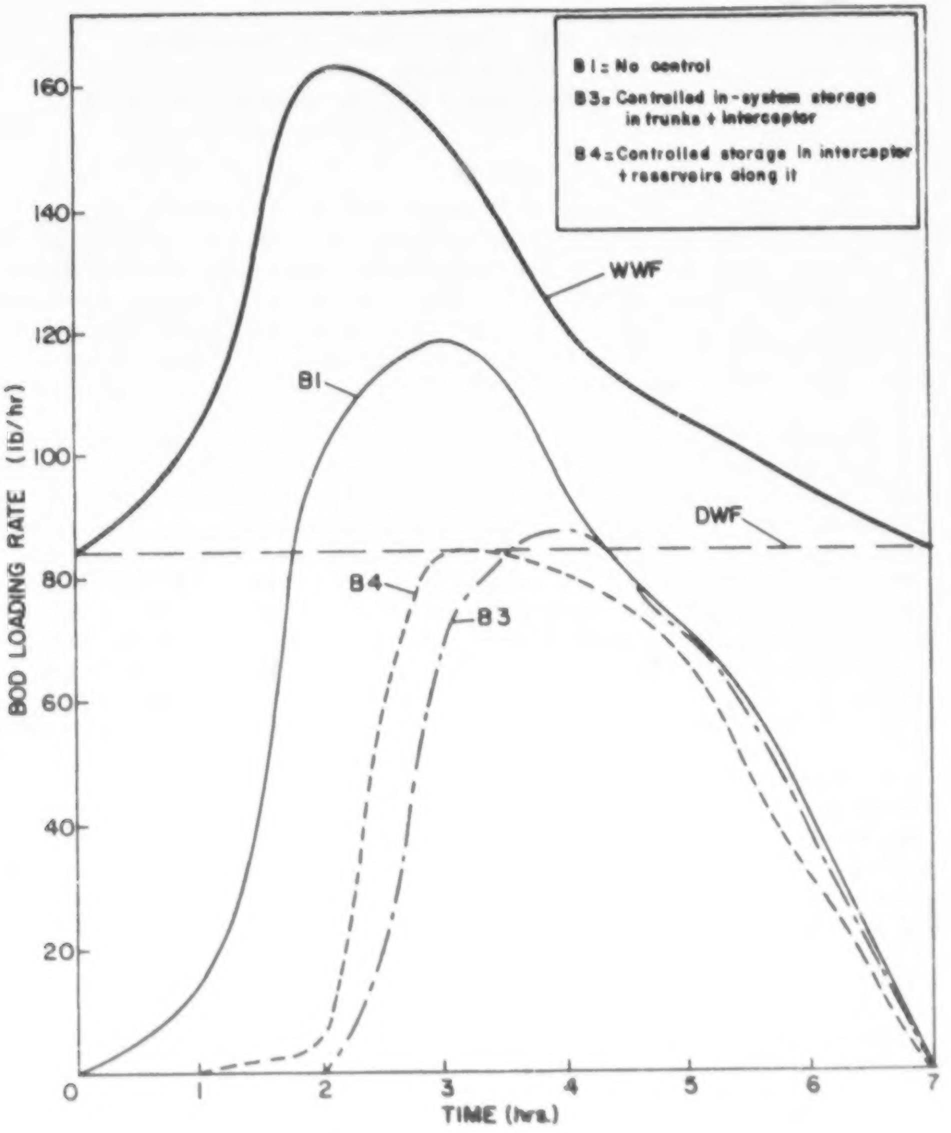


FIG. 9 OVERFLOW BOD POLLUTOGRAPH

Methodology for Evaluating Agricultural Best Management Practices (BMP's)

by

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The goals of the Water Pollution Control Act Amendments of 1972 (FWPCA, PL 92-500) will be achieved only if nonpoint source, as well as point source, pollution is controlled. Authority exists under the FWPCA and the Clean Water Act of 1977 (PL 95-217) for the Environmental Protection Agency (EPA), in conjunction with individual states, to devise policies and initiate control programs to manage nonpoint source pollution. Progress, however, has been slow due to various reasons.

Based on a feasibility study undertaken by Meta Systems Inc for the EPA (September, 1978), a policy analysis methodology is presented here that addresses the problem of developing and evaluating effective strategies for the management of agricultural nonpoint source pollution. Figure 1 indicates that the proposed methodology is specifically designed to assess both the instream water quality and socio-economic impacts of agricultural practices and specific government policies aimed at encouraging agricultural practices which decrease nonpoint source pollution. Thus the methodology allows the simultaneous examination of 1) the water quality impacts of selected agricultural practices in the watershed and downstream waterbodies and 2) the economic effects (upstream and downstream) that alternative practices and nonpoint source pollution control policies have upon the farmer and downstream user. At this point the nonpoint source pollution control problems which the methodology addresses are limited to those that are amenable to solutions by incremental on-farm adjustments for damage reduction. Such problems would include, for example, the generation and transport of sediment and nutrients.

In developing the methodology, it was felt that potential methods for the assessment of environmental and socio-economic impacts of agricultural practices would be effective only if they exhibit the following characteristics.

- Compatibility between data availability and requirements
- Robustness under a wide range of alternative agricultural futures
- Capability to evaluate major policy options
- Ease of Understanding
- Usefulness at the appropriate planning level
- Applicability to the full range of on-farm options

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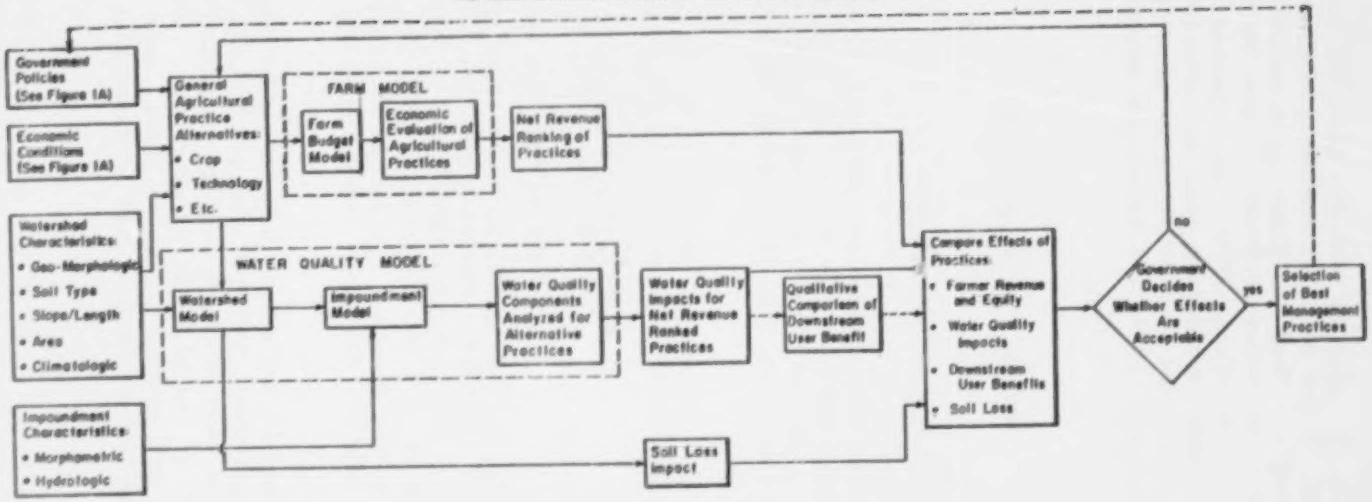
*This paper was presented by Dr. Jochen Kühner at EPA's SWMM Users Group Meeting in Annapolis, Maryland on November 13, 1978.

FIGURE 1

METHODOLOGY FOR ASSESSMENT OF WATER QUALITY
AND SOCIO-ECONOMIC IMPACTS OF AGRICULTURAL PRACTICES


Government Policies (See Figure 1A)
Economic Conditions (See Figure 1A)
Watershed Characteristics:
• Geo-Morphologic
• Soil Type
• Slope/Length
• Area
• Climatologic
Impoundment Characteristics:
• Morphometric
• Hydrologic
General Agricultural Practice Alternatives:
• Crop
• Technology
• Etc.
FARM MODEL
Farm Budget Model
Economic Evaluation of Agricultural Practices
Net Revenue Ranking of Practices
WATER QUALITY MODEL
Watershed Model
Impoundment Model
Water Quality Components Analyzed for Alternative Practices
Water Quality Impacts for Net Revenue Ranked Practices
Qualitative Comparison of Downstream User Benefit
Soil Loss Impact
Compare Effects of Practices:
• Farmer Revenue and Equity
• Water Quality Impacts
• Downstream User Benefits
• Soil Loss
Government Decides Whether Effects Are Acceptable
no
yes
Selection of Best Management Practices

In the present study it appeared that the best way to evaluate the proposed methodology on the basis of the above characteristics was to work through an example. Thus the Black Creek watershed in northeastern Indiana, a USEPA/USDA demonstration project (see Christiansen and Wilson, 1976; Lake and Morrison, 1975), was selected and combined with a synthesized downstream impoundment with characteristics typical of those found in the Corn Belt (for details, see Meta Systems Inc, September, 1978; Wineman et al., 1978, and Walker and Kühner, 1978a and 1978b).

Methodology Development

Agricultural Practices and the Farm Model

While major market and regulatory pressures -- such as prices, taxes, subsidies, government regulations -- are exerted at a regional or national level, it is the farmer who responds by choosing both his crops and methods of farming. For this reason the methodology has to include a farm budget.

It is assumed that the farmer desires to maximize net revenues. Accordingly, he chooses a set of agricultural practices such as:

- crop rotation
- tillage practices
- structural erosion and drainage control practices; and
- levels of chemical application

These choices are represented as inputs to the farm model for the calculation of a variety of costs associated with operating the farm in the specified manner.* This required developing a data base for the model. Each element of cost was updated for 1977 prices and modified where necessary to adapt the model for the Black Creek area. Additional inputs to the model specify expected yields and market prices for each crop.

In the case study 11 practices (plus two modifications to include custom hiring) are selected, and farm budgets are set up for uniform farms of 250 acres on each of three soil types (upland, ridge, and lowland) characteristic of the Black Creek watershed. Table 1 shows the revenue, costs, and net returns for three farms, each on one soil type, assuming uniform adoption of each of the 11 farm practices and existing government policies.

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*The procedure described by Dr. Klaus Alt in Appendix C of Control of Water Pollution from Cropland (USDA, U.S. EPA, 1976) was applied here.

TABLE 1. SUMMARY OF FARM MODEL OUTPUT - 1977 DOLLARS, IN THOUSANDS
(UNDER EXISTING GOVERNMENT POLICIES)

Farm Practice / Revenue and Cost	Tillage Practices			Rotations							Terraces		
	corn, conven-tional tillage (CC-CV)	corn, chisel plow (CC-CH)	corn, no-till (CC-NT)	corn soybean, conven-tional tillage (CB-CV)	corn soybean, chisel plow (CB-CH)	corn soybean no-till (CB-NT)	corn, soy-bean, wheat, hay, partial use of herbicides (CBWH)*	(CBWH)	corn, soy-bean, wheat hay, no-till (CBWH*-NT)	(CBWH-NT)	corn conven-tional tillage (CC-CVT)	corn chisel plow CC-CHT	corn soy-bean no-till (CB-NTT)
GROSS REVENUE													
A. Upland soil	52.5	52.5	49.9	46.3	46.3	44.4	43.0	43.0	43.0	43.0	56.0	56.0	47.4
B. Ridge soil	65.0	65.0	65.0	59.1	59.1	57.9	51.8	51.8	51.8	51.8	68.5	68.5	60.9
C. Lowland soil	65.0	65.0	52.0	59.1	59.1	50.7	49.9	49.9	49.0	49.0	68.5	68.5	53.7
COSTS													
A. Upland soil	39.7	39.1	43.0	32.9	32.6	32.3	34.4	30.6	34.2	30.3	46.4	45.8	38.9
B. Ridge soil	41.4	40.9	44.9	33.3	33.1	32.7	34.7	31.0	34.4	30.7	48.2	47.6	39.3
C. Lowland soil	42.7	42.1	45.5	34.8	34.5	34.1	35.4	31.7	35.1	31.4	49.4	48.9	40.7
NET RETURN													
A. Upland soil	12.8	13.4	6.9	13.5	13.7	12.2	8.5	12.4	8.8	12.8	9.6	10.2	8.6
B. Ridge soil	23.6	24.1	20.1	25.8	26.1	25.1	17.4	20.8	17.4	21.1	20.3	20.9	21.5
C. Lowland soil	22.3	22.9	6.5	24.4	24.6	16.6	14.5	18.1	13.9	17.6	19.1	19.6	13.0

Columns may not add due to Rounding.

Water Quality Model

Examples of watershed (practice/soil type combinations) and water quality analyses are based on the assumption of a homogeneous watershed (as assumed in the farm model). At this preliminary stage of methodology development this approach was considered to be more appropriate than one dealing with aggregate economic and environmental impacts in a heterogeneous watershed (which would incorporate variations in soil, slopes, farm sizes, and other characteristics).

The water quality analysis is separated into two major sections:

1) the watershed, which is characterized as generating different loadings of pollutants depending on agricultural activities and watershed characteristics.
2) the impoundment, where water quality is dependent on the type and quantity of loadings from the watershed and on impoundment characteristics.

In this scheme the river is represented as a medium for transporting pollutant loadings from the watershed to the impoundment. Water quality conditions in the river reflect these loadings, which enter the river in surface runoff and groundwater base flow and are transported in dissolved and sediment-bound phases.

The methods developed for the watershed analysis are of an empirical nature and are concerned with long-term average emissions (though modified, they are consistent with the mode of the Universal Soil Loss Equation (Wischmeier and Smith, 1972)). Although the use of long-term average time scales precludes direct assessment of responses under extreme meteorologic conditions, effects of the timing of various agricultural operations, seasonal variations in water quality, and analysis of relatively short-lived compounds, we feel that given currently available data and limited knowledge of the relevant physical processes, a framework built from complex models would generally not be feasible or useful at the regional planning level.

Pathways involved in the watershed analysis and impoundment water quality analyses are summarized respectively in Figures 2 and 3. Average annual export (delivery) rates of the following substances are evaluated in the watershed analysis: 1) sediment (sand, silt, and clay fractions); 2) phosphorus (biologically available); 3) dissolved nitrogen; and 4) dissolved color. The computed concentrations of these components are assumed to be representative of average water quality conditions in rivers draining the agricultural watershed. These results of the watershed analysis can be linked with downstream empirical models for the purpose of predicting quality impacts in impounded waters under steady-state, seasonal, or long-term-average conditions. As Figure 3 indicates, water quality components considered are suspended solids, phosphorus, nitrogen, transparency, and epilimnetic chlorophyll-a concentration.

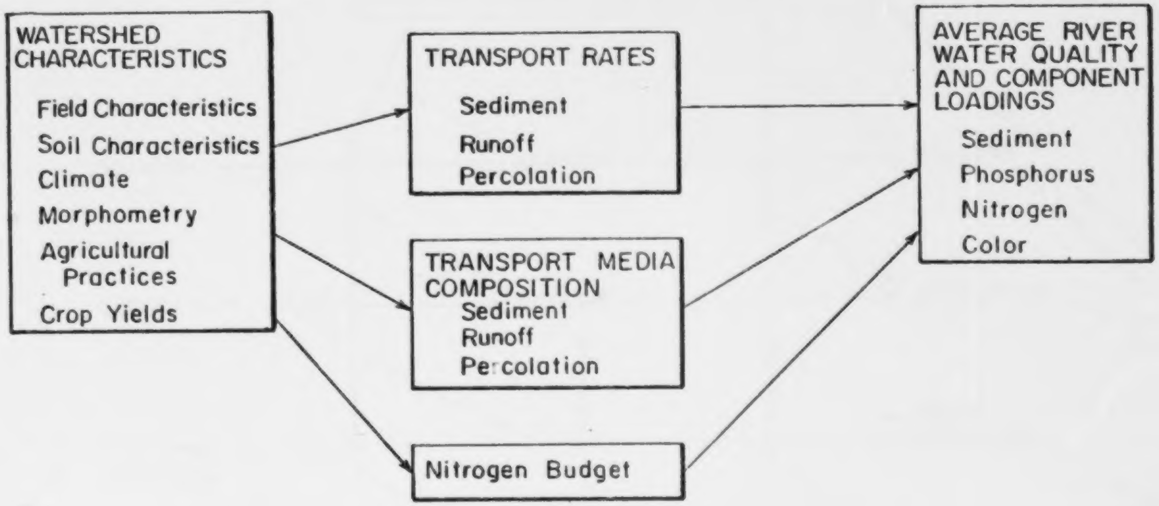
WATERSHED CHARACTERISTICS
Field Characteristics
Soil Characteristics
Climate
Morphometry
Agricultural Practices
Crop Yields
TRANSPORT RATES
Sediment
Runoff
Percolation
TRANSPORT MEDIA COMPOSITION
Sediment
Runoff
Percolation
Nitrogen Budget
AVERAGE RIVER WATER QUALITY AND COMPONENT LOADINGS
Sediment
Phosphorus
Nitrogen
Color

Figure 2. Pathways in the Watershed Analysis

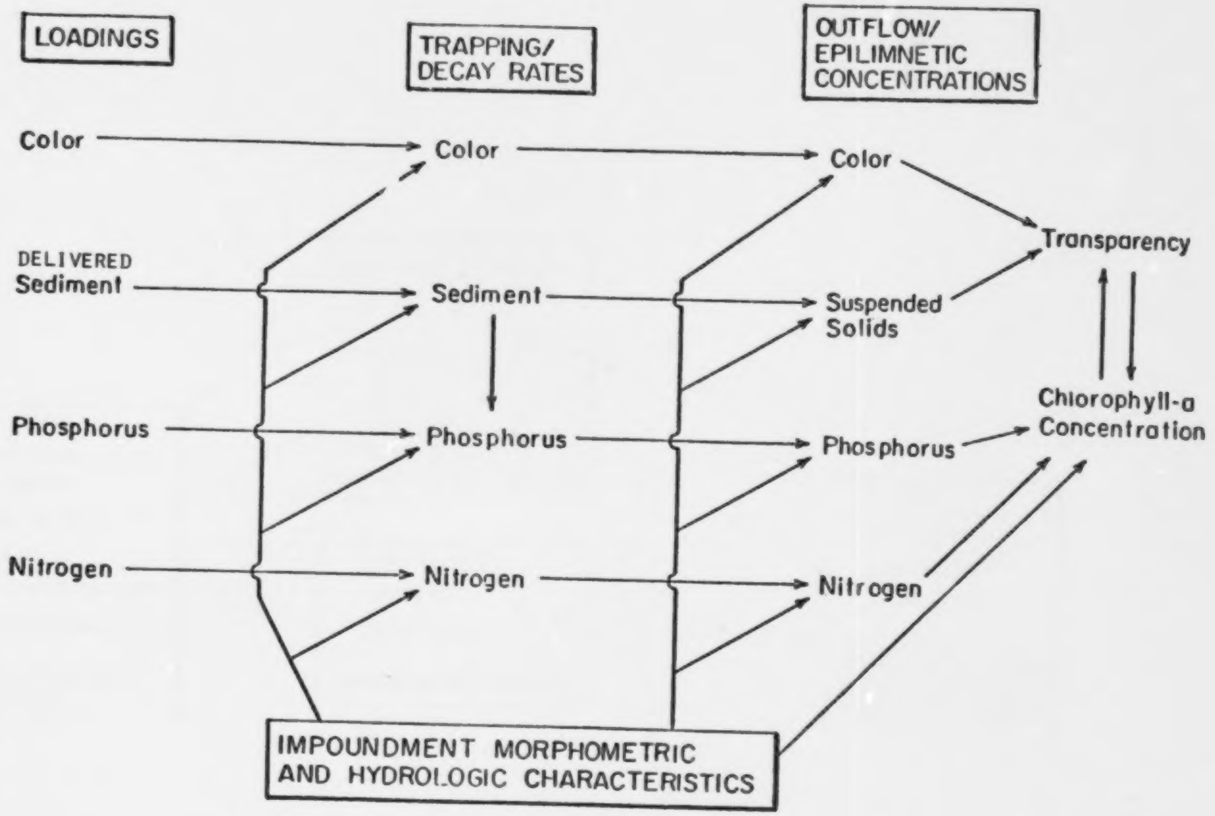
LOADINGS
TRAPPING/ DECAY RATES
OUTFLOW/ EPILIMNETIC CONCENTRATIONS
Color
DELIVERED Sediment
Phosphorus
Nitrogen
Color
Sediment
Phosphorus
Nitrogen
Color
Suspended Solids
Phosphorus
Nitrogen
Transparency
Chlorophyll-a Concentration
IMPOUNDMENT MORPHOMETRIC AND HYDROLOGIC CHARACTERISTICS

FIGURE 3. PATHWAYS IN THE IMPOUNDMENT WATER QUALITY ANALYSIS

The latter is used as an index of eutrophication and represents the extent of the algal growth in the surface waters of impoundments during the summer. The model developed for predicting chlorophyll-a levels considers the possible limitation of algal growth by light, phosphorus, and/or nitrogen. Transparency is computed as a function of suspended solids, dissolved color, and chlorophyll-a concentrations. Models are formulated for each of the above water quality components based upon theoretical considerations and the results of previous modeling efforts. They are calibrated and tested empirically using a data base characterizing the behavior of these components in Corn Belt impoundments and compiled from various sources (EPA/NES, 1975; USDA, 1969; ISBH, 1976; USACE, 1977).

Important additional independent variables include total watershed area, impoundment surface area, and impoundment mean depth. Assumed values for the examples discussed below are 200 km^2, 5 km^2, and 4 meters respectively. It should be noted that the evaluations of the relative impacts of the agricultural practices on impoundment water quality may be somewhat sensitive to the choice of the watershed/impoundment configuration.

Impacts on Downstream Users

To estimate the impacts of alternative farming practice/soil type combinations on downstream users, changes in water quality must be related to measurements of value to these users. Depending on the use of the water and the land uses surrounding the watershed and impoundment, certain water quality components are of more or less inteest to different groups of people concerned with water quality (users).

Some benefit categories of interest in this case are human health, municipal water supply, flood control, ecology, recreation, aesthetics, and the local economy. Rigorous quantitative methods of benefit estimation would vary depending upon the particular benefit category of interest (see Appendix E of Meta Systems Inc, September, 1978). It is clear that a comprehensive benefit estimation methodology covering several such measurement techniques would be a major undertaking. As an alternative appropriate for policy-making and agency resources, a simplified version is presented here that qualitatively assesses the direction of benefits (as perceived by each user group) resulting from water quality changes induced by the alternative practices. As the example (table 2) shows, a minus sign has been chosen to indicate that an increase in a specific water quality component has a detrimental effect on the specified benefit group; for example, an increase in nitrogen concentration in drinking water is potentially harmful to human health. A zero indicates that an increase in the parameter is of no importance to the benefit category. For instance, the same increase in nitrogen concentration just mentioned would not impact flood control operations in the impoundment. A water quality component increase which has a positive impact on a benefit category is indicated by a plus sign. Increasing impoundment biomass, for example, might improve sport fishing since more food might increase the available fish population.

Use of the Farm and Water Quality Models

Comparison of Practices

The methodology described above can be used to illustrate: 1) how agricultural practices can be evaluated in terms of water quality impact to facilitate selection of BMP's (under existing government policies); 2) how government policies which encourage the implementation of practices that are conducive to water quality improvements (i.e., BMP's) may be examined; and 3) how changes in economic conditions can affect agricultural practices and water quality impacts.

The evaluation of agricultural practices under current policies uses the 11 selected farm practices listed (and defined) in Table 1 (as if they constituted a comprehensive set of alternatives currently available to farmers). In addition to gross soil loss (i.e., erosion), six variables related to water quality were analyzed for the three soil types (upland, ridge, lowland) and the 11 farm practices. For illustrative purposes, the results (together with net revenues) for one soil type, the lowland, are displayed as a set of bar graphs (Figure 4). In the graph increasing pollutant loads or concentrations are shown by higher vertical height of the bar; for net revenue vertical height increases with higher returns.

In terms of net revenue ranking of the 11 farm practices, the corn-soybean rotation is most profitable based on prices chosen for these commodities in this example (i.e., corn, \$2.00 per bushel; soybeans, \$5.00 per bushel; wheat, \$2.50 per bushel; hay, \$60 per ton). This practice is also most profitable on the other two soil types.

Figure 4 can also be used to examine the differences in soil loss (gross erosion) or any of the six water quality components for each farm practice on each of the three soil types. For example, on the lowland, the practice that maximizes net revenue (\$24,600) is calculated to produce an annual gross soil loss of 0.49 kg per square meter* (gross losses range from 0. 78 to 0.1 kg/m^2 before application of sediment delivery ratios). This can be contrasted with results obtained for the other two soil types. On the ridge the annual net revenue maximizing (\$26,100) practice results in 1.2 kg/m^2 annual gross soil loss (range: 2.1 to 0.2 kg/m^2). On the upland the net revenue maximizing (\$13,700) practice shows an annual soil loss of 3.4 kg/m^2 (range: 6.1 to 0.6 kg/m^2). On all soil types the no tillage corn-soybean-wheat-hay rotation (CBWH-NT) minimizes gross soil loss.

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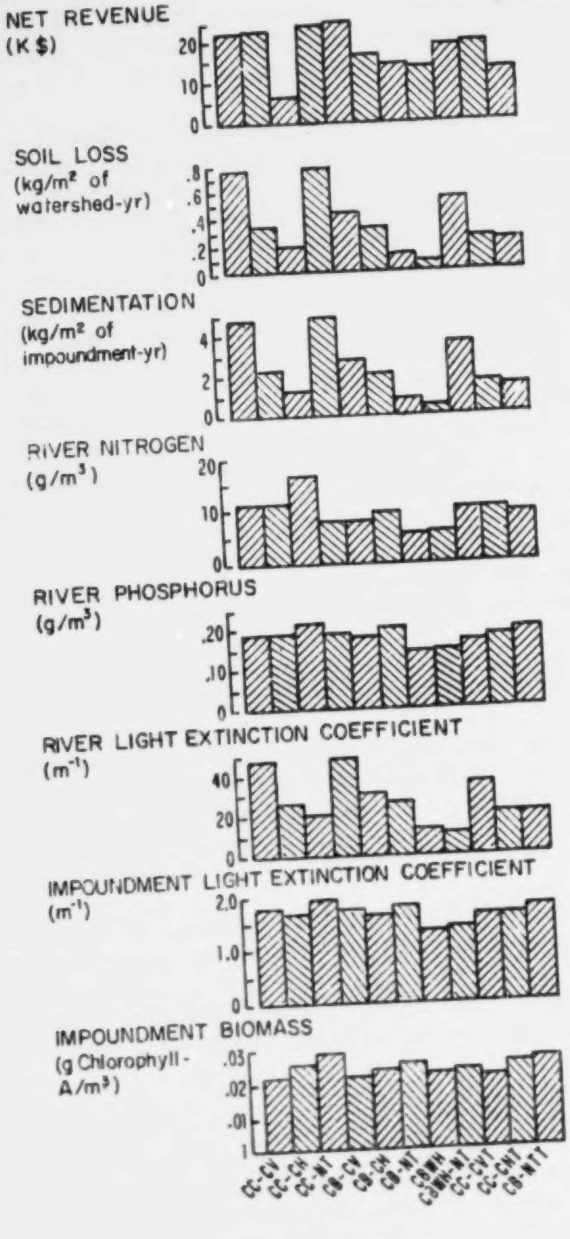
*kg/m^2 = 4.5 tons/acre.

Figure 4. Comparison of Practices -- Lowland (see Table 1 for definition of practices)



The water quality impacts of agricultural practices vary with field/soil type, water body (river versus impoundment), and specific pollutant. Use of gross soil loss alone as the criterion for farm practice evaluations can lead to erroneous conclusions because of the importance of various dissolved components and the interactive effects of different processes (e.g., decay, adsorption/desorption, sedimentation).

The types of gross soil loss (erosion) controls evaluated here do not appear to have proportionate imapcts on phosphorus losses and impoundment eutrophication problems. This results from the influence of the following factors which are considered in the predictive (modeling) framework: 1) a small fraction of soil phosphorus (five to ten percent) is biologically available; 2) minimum tillage methods tend to cause phosphorus enrichment of the surface soil layer and create a potential for leaching of phosphorus from crop residues during snowmelt periods; 3) impoundment phosphorus trapping efficiency is correlated with sedimentation rate; 4) reduced erosion rates result in increased availability of light to promote algal growth in downstream impoundments. Because of the combined effects of these factors, the agricultural practices evaluated do not appear to be effective in controlling eutrophication and in some cases cause moderate increases in algal growth (see Figure 4). In such cases corresponding increases in the potential for fish production could be interpreted as water quality benefits, although additional analysis would be needed in order to quantify such benefits. Impacts of soil loss controls on suspended solids concentrations and transparency levels are generally in the same direction as impacts on gross erosion rates, but are usually less. The results are specific to the field/soil types examined and to the assumed watershed/impoundment configuration.

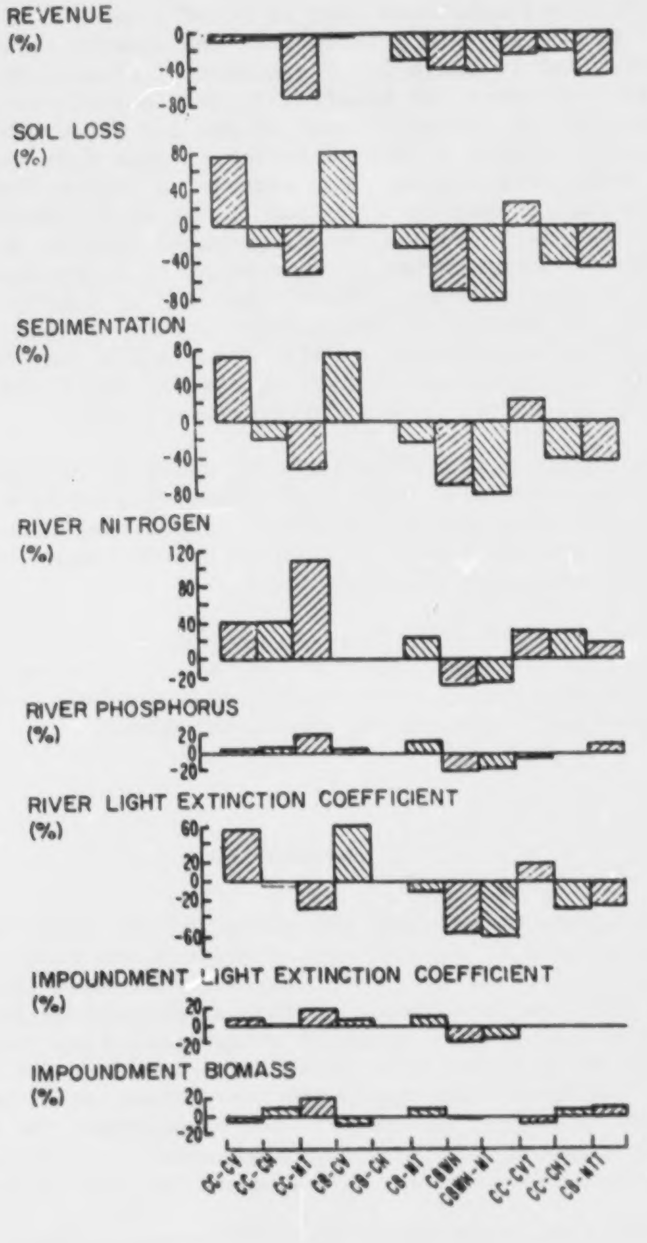
The importance of one pollutant compared to another may also shift from watershed to watershed and hence influence the selection of those water quantity components of primary importance to the evaluation of the BMP's. In assessing BMP's it seems reasonable to rank the different pollutants on the basis of severity of local water quality issues.

Our report (Meta Systems Inc, September, 1978) shows that the methodology can be used equally well for the other purposes mentioned above. Policy examples investigated in the study include prohibition of certain cultivation practices, gross soil loss restrictions, and fertilizer control policies. Changing economic conditions are evaluated using an example of an energy future.

Downstream User Effects

In order to compare the various agricultural practices from the downstream users' point of view, we need to select a base case. Having assumed that the farmer is a maximizer of net revenue, we choose the practice producing the highest net revenue (the corn-soybean rotation using chisel plowing (CB-CH)). To illustrate, Figure 5 depicts the relative water quality, gross soil loss, and net revenue impacts (measured as percentage increases or decreases relative to the base case) of the other ten practices on the lowland soil type. As described above, the downstream

Figure 5. Percent Change of Highest Revenue Factor -- Lowland



benefits of alternative farming practices can be qualitatively compared by mapping the quantitative practice/water quality relationships depicted in Figure 5 onto each user or benefit group. The resulting relative impacts of the 11 farm practices on the benefit categories of interest are summarized in Table 2. The number of pluses, minuses, or zeros in each cell indicates the beneficial, detrimental, or neutral impact respectively of the change in each of the six water quality components, displayed in Figure 5, when a switch is made from the base case (CB-CH) to the compared practice. For example, a switch from the base case to a corn-soybean-wheat-hay rotation (CBWH) would result in a decrease in each of the six water quality components, ranging from a decrease in impoundment sedimentation of 70 percent to a decrease in impoundment biomass of three percent (see Figure 5). It can be seen from the six pluses in the second row and seventh column of Table 2 that the benefit category, municipal water supply, for example, has been impacted positively by the decreases in each of the six water quality components caused by the switch in agricultural practices.

The analysis also shows that in general, with the possible exception of the beneficial impact of higher biomass levels on sport fishing, no conflicts exist among the benefit categories; i.e., all categories are either not influenced or negatively influenced by an increase in any of the water quality components.

In contrast to many other case studies, no attempt has been made here to weigh the water quality components or benefit categories, since this would introduce complex aggregation problems and difficult value judgments highly specific to individual cases.

Conclusion

A proposed methodology for assessing the water quality and socioeconomic impacts of agricultural practices has been described and tested in a case study, and its potential uses have been illustrated. It appears that the development of such a methodology for regional and state level planning for nonpoint source pollution control is feasible and would be of significant value for broad analyses of large numbers of policy alternatives, including identification of best management practices. However, as noted in the discussion, the methodology is currently at a preliminary stage of development and further refinements, such as those summarized below, are necessary to make it fully operational.

- While the farm budget model as presented here captures the major elements important for assessing the economic impacts of alternative nonpoint source pollution control policies on the farmer, further modifications would be necessary before it could be used effectively in a planning context. Most important, the model should

Table 2. Summary of Relative Impacts of Farming Practices on Benefit Categories

Soil Type: Lowland	Farming Practices										
Benefit Categories	CC-CV	CC-CH	CC-NT	CB-CV	CB-CH	CB-NT	CBWH	CBWH-NT	CC-CVT	CC-CHT	CC-NTT
human health (drinking water)	1(+) 1(0) 4(-)	2(+) 1(0) 3(-)	2(+) 1(0) 3(-)	1(+) 2(0) 3(-)	6(0)	2(+) 1(0) 3(-)	5(+) 1(0)	4(+) 2(0)	1(+) 2(0) 3(-)	2(+) 2(0) 3(-)	2(+) 1(0) 2(-)
municipal water supply	1(+) 5(-)	3(+) 3(-)	3(+) 3(-)	1(+) 1(0) 4(-)	6(0)	3(+) 3(-)	6(+)	5(+) 1(0)	1(+) 1(0) 4(-)	3(+) 1(0) 1(-)	3(+) 3(-)
dredging (flood control)	5(0) 1(-)	1(+) 5(0)	1(+) 5(0)	5(0) 1(-)	6(0)	1(+) 5(0)	1(+) 5(0)	1(+) 5(0)	5(0) 1(-)	1(+) 5(0)	1(+) 5(0)
ecology	1(+) 5(-)	3(+) 3(-)	3(+) 3(-)	1(+) 1(0) 4(-)	6(0)	3(+) 3(-)	6(+)	5(+) 1(0)	1(+) 1(0) 4(-)	3(+) 1(0) 2(-)	3(+) 3(-)
recreation sport fishing	1(0) 5(-)	4(+) 1(0) 1(-)	4(+) 1(0) 1(-)	1(0) 5(-)	6(0)	4(+) 1(0) 1(-)	4(+) 1(0) 1(-)	4(+) 2(0)	2(0) 4(-)	4(+) 2(0)	4(+) 1(0) 1(-)
contact	1(-) 1(0) 4(-)	2(+) 1(0) 3(-)	2(+) 1(0) 3(-)	1(+) 2(0) 3(-)	6(0)	2(+) 1(0) 3(-)	5(+) 1(0)	4(+) 2(0)	1(+) 2(0) 3(-)	2(+) 2(0) 2(-)	2(+) 1(0) 3(-)
non-contact	1(+) 2(0) 3(-)	2(+) 2(0) 2(-)	2(+) 2(0) 2(-)	1(+) 2(0) 3(-)	6(0)	2(+) 2(0) 2(-)	4(+) 2(0)	3(+) 3(0)	1(+) 3(0) 2(-)	2(+) 3(0) 1(-)	2(+) 2(0) 2(-)
aesthetics	1(+) 2(0) 3(-)	2(+) 2(0) 2(-)	2(+) 2(0) 2(-)	1(+) 2(0) 3(-)	6(0)	2(+) 2(0) 2(-)	4(+) 2(0)	3(+) 3(0)	1(+) 3(0) 2(-)	2(+) 3(0) 1(-)	2(+) 2(0) 2(-)
local economy	1(+) 5(-)	3(+) 3(-)	3(+) 3(-)	1(+) 1(0) 4(-)	6(0)	3(+) 3(-)	6(+)	5(+) 1(0)	1(+) 1(0) 4(-)	3(+) 1(0) 2(-)	3(+) 3(-)

be automated, perhaps employing a revenue-maximizing linear programming model for policy analysis. In addition, a much broader range of agricultural practices must be considered, including variations in fertilizer applications, organic farming, and livestock integration.

- While the model includes what we consider to be those functional aspects of the watershed/impoundment system which are required for evaluation of the impacts of farm management practices on water quality, it requires additional analysis, calibration, and testing before it can be applied in a planning context. A preliminary sensitivity analysis has helped to identify those aspects of the model framework which are most critical to the predictions (See Meta Systems Inc, (Appendix D), September, 1978).

- Additional water quality components that must be included in the framework are dissolved oxygen, biocides, and biocide residues.

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*Appendix A, Farm Model.
Appendix B, Methods for Predicting Watershed Loadings.
Appendix C, Methods for Predicting Impoundment Water Quality.
Appendix D, Water Quality Impact Results: Additional Interpretations and Sensitivity Analysis.
Appendix E, A Discussion of Benefit Estimation.
Appendix F, Crop Response to Fertilizer.

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COMPUTER SIMULATION OF FLOOD RELIEF WORKS UTILIZING INLET CONTROL AND DETENTION STORAGE

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COMPUTER SIMULATION OF FLOOD RELIEF WORKS UTILIZING INLET CONTROL AND DETENTION STORAGE

INTRODUCTION

Basement flooding as well as pollution from combined sewer overflows have become a very serious problem for many urban areas throughout the world.

The damage that results is staggering. For example, within Metropolitan Chicago, the Corps of Engineers have estimated that about 400,000 buildings are subject to flooding, causing an average annual damage of $280 million. In addition, pollution of streams, rivers and Lake Michigan cause a further estimated damage of about $500 million annually. Chicago may be the most effected area in North America, but the problem is widespread and affects us all, either as a suffering homeowner, or as citizens having, through taxes, to participate in the cost of providing relief measures.

Nor is the problem related only to combined sewers; more serious damage has occurred in areas where foundation drains have been connected by gravity to separate storm sewers, a practice used by a number of municipalities in Canada. In such cases, a hydrostatic pressure will be created on basements during times when the storm sewer surcharges as a result of runoff exceeding the sewer capacity. When the level of surcharge exceeds the floor level of any adjacent basement by one foot or even less, *the uplift created is sufficient to cause basement floors to heave and crack.* Not only can such pressure cause severe structural damage to a house, but the inflow of storm water through a cracked

INTRODUCTION - cont...

floor will enter the sanitary sewer system through the basement floor drains, causing sanitary sewer back-up as well. Damage of this type is well documented, but unfortunately it is rarely covered by insurance, leaving the homeowners to pay the cost of repairs.

The cause of the problem is easy to understand; our sewers simply do not have sufficient capacity to handle every runoff eventuality. To provide that would not only be very costly indeed, but also result in moving problems downstream often causing more serious damage. *The past policy of removing storm water as fast as possible has resulted in very severe damages not only to downstream areas, but to the environment as a whole. We have tried to ignore nature and have come out as losers.*

CONVENTIONAL SOLUTION

The conventional solution nevertheless has always been to provide more sewer capacity by constructing relief sewers or tunnels. This method has many shortcomings; in spite of the high costs, it generally is not very effective in providing relief for areas not directly served by the relief sewers. Due to the very high costs, we cannot afford to install relief sewers on all the local streets and as a result areas upstream of the relief sewers will generally not receive significant flood relief.

Sewer separation must, by necessity, commence from an adequate downstream outlet. Quite often, however, the most frequent flooding problems occur during high intensity-short duration storms, i.e. summer thunder storms. Such rainfall events result in large volumes of runoff in a relatively short period of time, causing overloading in upstream areas, whereas downstream areas adjacent to trunk sewers with larger conduit capacity are not as susceptible. To provide relief by conventional methods, relief sewers therefore have to be extended into the upstream areas also.

ALTERNATIVE SOLUTION

The solutions our firm has employed for solving drainage problems in existing built-up areas were derived by approaching the problem *from the exact opposite direction*. Since existing sewer systems usually will handle all flows except during the most intensive storms, we concluded that *it would be more appropriate if the inlet capacity could be controlled, so that the rate of inflow would not exceed the capacity of the existing sewer system*. We have termed this solution the INLET CONTROL METHOD.

Storm water detention along with inlet control does not only lend itself as a means of solving drainage problems in existing areas, but it is also suitable for new areas, particularly where zero increase in peak runoff is required to prevent environmental damage or where treatment of storm water runoff would otherwise be required to maintain pollution at acceptable levels. Based on our research and experience to date, we are confident that in many such cases a combined sewer system with inlet control and source detention will give the highest value-effective solution.

The benefits of using inlet control and detention storage for either combined or separate systems can be briefly summarized as follows:

1. In areas with combined sewers:
    - eliminates the need for costly sewer separation on streets and private property.
    - can provide immediate relief in areas with most severe flooding problems, leaving less prone areas until funds become available.
    - provides a level of protection much higher than what usually is available by conventional methods, and likely at much lower costs.
    - permits polluted storm water ("first flush") to be treated by existing treatment facilities.
    - prevents pollution from combined sewer overflows.
    - reduces peak flows in receiving waters, resulting in less flooding damage and stream erosion downstream.

ALTERNATIVE SOLUTION - cont...

2. In areas with separate storm sewers, but with foundation drains connected by gravity, it further
   - eliminates structural and flooding damage to basements during times when the rate of runoff exceeds the sewer capacity.

The inlet control method has become more practical with the introduction of the "Hydro-Brake" regulator. A Hydro-Brake is a patented flow controller made of stainless steel. It is self regulating and has no moving parts. It requires no power, but uses the static head of stored water to create its own "energy" to retard the flow. The movement of water through a Hydro-Brake involves a swirl action, dissipating energy to control the rate of discharge.

Although the function of a Hydro-Brake is somewhat similar to an orifice, it has certain important advantages:

1. It permits a much larger opening for passage of the same amount of water. This is particularly important where clogging is a possibility, such as for instance in catchbasins. It is also important where sanitary or combined sewage flows are being regulated.

2. The flow rate of a Hydro-Brake is not significantly affected by a variation in head. This is important where it is desirable to maintain a relatively large passage for the water, yet also maintain a fixed maximum rate of flow during peak conditions.

3. The outflow from a Hydro-Brake does not create a high velocity jet stream as an orifice will, thus avoiding scouring inside sewer pipes.

Three case studies will be presented here, where the inlet control method and detention storage have been designed with the aid of computer modelling, to alleviate flooding problems in existing urban areas in three separate municipalities in the Toronto area.

BOROUGH OF YORK

The Borough of York, being an older municipality, is served almost exclusively by combined sewers which were originally designed to accommodate a 2 year design storm, but due to increased imperviousness and physical weaknesses of the sewer system, now in many cases have much less capacity. Further aggrevating the problem are the facts that stormwater from roofs and ground water from foundation drains along with the sanitary service are connected to the sewer. The result has been more and more frequent flooding of basements. Studies carried out about 10 to 12 years ago recommended sewer separation, and for about eight years the Borough, with a budget of $1 million annually, proceeded to do just that. With the cost rapidly escalating, it became apparent that adequate relief would not be in sight for many years without over-extending the Borough's ability to pay.

In 1976 our firm was engaged by the Borough of York to carry out a study with the objective of finding alternative solutions for solving the flooding problems, which were most pronounced in four specific areas.

The study was to be based on providing relief to accommodate 2 year design storms without causing surcharge above existing basement floors. This criteria has been a standard for many years, not only in the Borough of York, but also in the adjoining City of Toronto. This, at our suggestion, was extended to include cost estimates for the 5 year and 10 year design storms as well.

BOROUGH OF YORK - cont...

Our study concluded that conventional methods of providing relief sewers would be unfeasible due to the fact that three of the four areas were located at a considerable distance from suitable storm water outlets. This made us search for alternative methods, of which the inlet control method and detention storage was found to be the most value effective by far. The inlet control was achieved by placing Hydro-Brake regulators in the system and by disconnecting downspouts or, where this is not feasible, placing flow restrictors in them.

Catchbasins were sealed where positive drainage could be achieved thus utilizing the major system to carry stormwater runoff. At low points where runoff would accumulate, storm sewers were provided to carry the water to detention tanks (see Fig.1). Stormwater would then be discharged into the combined sewer at a predetermined release rate, such that the combination of roof runoff, surface runoff plus dry weather flows would not cause flooding damage.

In this particular case, the majority of storage was accommodated in subsurface tanks due to the high urban density and lack of open space which is characteristic of many older sections of the city.

The preliminary estimates prepared as part of the study for the remedial works required for the four areas indicate a range of costs as follows:

2 year design storm	$110,000
5 year design storm	$285,000
10 year design storm	$830,000

As a result of the findings of our study, the Borough decided to proceed on the basis of providing protection against 10 year storms and to have the work installed for three of the four areas as early as possible. The Borough was particularly pleased with the fact that the flooding

MODEL SELECTION - cont...

The program pays special attention to the unsteady character of all the flow with respect to time, the retention capacity of the watershed surface and sewer networks, the flow regulating effect of backwater and the inter-connective effect of sewer branchings. Flow retardation and diminishing processes in a watershed during a storm are simulated by the surface runoff model which transforms the hyetograph into individual surface runoff hydrographs. This model enabled us to simulate both the minor and major systems simultaneously, along with the flow regulators and detention facilities.

MODELLING

For each catchment area the existing sewer system carrying dry weather flow plus a percentage of roof flow (the minor system) and the overland drainage system (the major system) were modelled simultaneously.

Initial storage volumes were estimated by modelling a restrictor located in the inlet from the road system to the sewer system with an imaginary free outlet at the obvert of the sewer.

Hydrographs produced at the imaginary outlets were integrated to produce the required storage volumes. After the preliminary volumes had been produced, storage tanks were inserted into the model and the overall system was simulated.

Downstream back water effects were also established by including the downstream areas in the simulation.

It was found that allowing unrestricted roof water entry into the combined problems in the most affected areas could be rectified relatively quickly and not have to wait for the sewer separation programme to be completed, and at costs within their annual sewer separation budget.

MODEL SELECTION

For this particular application a model would have to possess the capability of effectively simulating numerous flow dividers, overflow weirs and many complex interconnections which are characteristic of modification in older urban areas. Backwater effects from downstream areas where flood relieve works were not to proceed would also have to be accounted for. In short, a very sophisticated sewer hydraulic model was required. The two models considered for this application were the SWMM model with the Extran option, and the Dorsch HVM model.

Since the adjoining municipality, the City of Toronto, into which our study areas outlet, was using the Dorsch HVM model,we decided to use the same, permitting us to utilize the City of Toronto's input and output results for the backwater effects.

The HVM model is a single event rainfall-runoff computer program package for detailed hydraulic analysis of existing and planned sewer networks. The program was developed by the German consulting firm Dorsch Consult in Munich over a decade ago, but is constantly being modified to meet the objectives of any particular project where it is to be applied.

The model incorporates the principles of continuity and energy (Saint Venant's) equations. These equations are transformed into a system of finite differential equations and modified by various mathematical manipulations. The overall solving procedure is provided by a repetitive downstream-upstream calculation scheme which involves further internal iterations and loops.

MODELLING - cont...

sewer while regulating only surface inflow would result in the system surcharging. To reduce the surcharge to a level below the lowest basements, it was found that to achieve a 10 year level of protection about 50% of the flow from the downspouts would have to be discharged to the surface.

IMPLEMENTATION

During the course of the detailed design, we found that the plans showing existing utilities were rather inaccurate and in fact did not include many of the utilities subsequently found. For this reason we re-estimated the cost of the works prior to tender call, which resulted in a final estimate of $987,633.00. Construction commenced in May 1978 and was essentially completed by October. The actual final construction cost including contingencies will be very close to this amount.

In order to maximize the efficienty of the Hydro-Brake and minimize the storage volume requirements, a special head-regulating chamber was designed. This design, as shown on Fig.2 results in the maximum permitted release rate to be obtained relatively early during the runoff event, with the result that the required storage volume will be reduced. The chamber should be located at low points on roadways where runoff accumulates, or surface runoff should be directed into the chamber via pipes. Should the depth of water in the tank exceed the water elevation in the chamber i.e. when inflow into the chamber subsides, the flap gate will automatically open and water from the tank will enter the chamber until the static heads equalize.

As simulated in the modelling analysis, 50% of the roof area was assumed

IMPLEMENTATION - cont...

to be disconnected. Legal and physical constraints did not allow us to proceed with the disconnection, thus regulators are to be placed in the downspouts which will have the same effect in dampening the peak flows from the roofs.

PERFORMANCE TO DATE

The first installation in one of the study areas was completed late in September of 1978. Although rainfall events this year have not been as intense as those experienced last year, intensities which had previously caused problems have not produced any flooding complaints.

In order to evaluate the performance of the system as installed, as well as calibrating the computer model used in the analysis, we have now taken steps to monitor the system. These results will enable us to verify or improve the design techniques for future installations.

BRIDLEWOOD COMMUNITY

This area which is about 150 acres, consisting primarily of single family homes, is located within the Bridlewood community of the Borough of Scarborough and has been plagued by severe floodings since it was built.

Development of this area commenced about 1962 with storm drainage designed solely on the basis of a piped system (minor system) with a theoretical capacity to handle storms expected to occur once every five years.
No provisions were made to prevent flooding when runoff in excess of sewer capacity would occur. Neither did the design include for an over-

land route (major system) to lead excess water away, but contained numerous sags on the streets where water ponded. The problem was further aggravated by the connection of roof drains and foundation drains by gravity to the storm sewer. Flooding and structural damage to basements thus occurred from the surcharge in the sewers when the runoff exceeded the capacity of the storm sewer.

Our basic recommendation was to control the amount of storm water that would be permitted to enter the existing storm sewer system (minor system) so that the sewers will not be surcharged during times when the runoff exceeds the capacity of the sewer. In order to effectively control storm water entry to the storm sewer system, most downspouts within the drainage area should be disconnected from the sewer and allowed to discharge to the surface, utilizing precast concrete splash pads. In this way, by placing regulators in the catchbasins, effective inlet control would be obtained.

In order to avoid restrictors with small outlets, we proposed to reduce the number required by sealing off many of the catchbasins not located in roadway sags. The restrictors are individually sized to prevent only flows up to, but not exceeding the full capacity of the particular sewer segment.

During times when runoff will exceed the capacity of the restricted catchbasins, the excess runoff will accummulate in the sags until it overtops the crest and then continue to flow downstream. If ponding depths at the sags exceed an acceptable level, an inverted siphon (see Fig. 3) would be installed to transport the excess storm water downstream.

In this manner a major system was developed for the area, making full use of detention storage in the roadways, yet assuring that the storm sewer system would not surcharge. The area fortunately benefits from having open space locations suitable for additional surface detention, with the result that only one small underground storage tank is required. The runoff in excess of the roadway detention will be conveyed to two low points in the area, from which it will flow through inverted siphons to a dry pond to be located on lands owned by the Hydro Electric Power Commission (see Fig. 4).

MODEL SELECTION

The minor system servicing the area consists of a simple tree type branching system, with no complex hydraulic structures. We based our initial design on the premise that downstream flood relief works were also to be initiated, such that the study area would be permitted to discharge at the gravity capacity of the downstream trunk sewer. The above two factors, along with the criteria that no storm sewer surcharging could be tolerated due to the connection of foundation drains, led us to select the Illinois Urban Drainage Area Simulator (ILLUDAS). Another capability of the ILLUDAS model which further enforced its suitability for this application was that this model can effectively simulate downspouts discharging to the ground.

The ILLUDAS model is based on the British Road Research Laboratory (RRL) model developed in the early sixties. Evaluation of the model suitability for urban areas in the United States was studied by Stall and Terstriep in the early seventies. Their findings indicated the RRL method provided an accurate method for predicting runoff from paved areas, but could not be recommended for all urban areas, unless runoff component from grassed areas was provided. The ILLUDAS model contains a grassed runoff component.

Each sub-basin may be discretized to three runoff producing areas; 1) paved areas directly connected to the storm drainage system; 2) grassed areas; 3) supplemental paved areas (paved areas discharging to ground surface). Hydrographs from the three runoff producing areas are developed for each sub-basin, combined and then become input into the drainage network at a particular point and routed downstream.

The model also contains other features which allow the modeller to input the soil type and antecedent moisture conditions. The model is also particularly suited for determining detention storage volumes, or outflow rate to make maximum use of available detention storage. Preliminary or final conduit sizing may be determined by the model, by executing it in the design mode. When the model is in the evaluation mode, flows exceeding the capacity of the conduit are stored at the upstream node.

MODELLING

A number of modifications were made to the ILLUDAS model to suite the objectives of our particular applications. These changes were made in consultation with M. L. Terstriep, co-author of the ILLUDAS model.

In order to simulate the effects of the roadway storage at the low points, the model was adjusted to account for storage in the rising limb of the hydrograph (see Fig. 5). The model was run in the design mode with one system representing both the sewer and the roadway system. Thus hydrographs were developed for the entire drainage system. The allowable sewer systems flow capacity was then subtrated from the hydrographs at each particular location, and the resultant peak road flow was obtained. Flow depths and ponding times for 2, 5, 25 and 50 year storms were then calculated manually and recorded (see Figs. 6, 7. 8, 9).

In order to simulate the dry pond in the H.E.P.C. lands, the allowable flow in the conduit was specified, thus the excess storm water was stored. With the conventional approach that ILLUDAS uses for storage (see Fig. 5) the specified discharge rate will be in effect until the storage volume is dissipated. In our application, the stored water is in off-line storage and will be released back into the system at a pre-selected release rate. In order to simulate this, the storage option was modified to allow for off-line detention storage as shown on Fig. 5.

The versatility and the relative low cost of using the ILLUDAS model allowed us to perform numerous computer runs for varying conditions and parameters, such as antecedent moisture conditions and soil classifications. We were thus readily able to evaluate the sensitivity of the system and arrive at the most cost-effective solution.

Although the Borough of Scarborough only requires sewers to accommodate 5 year design storms, our recommended solution has been designed to accommodate 50 year storms. The cost of the works which will be beneficial to 650 homes has been estimated to be about $400,000.

PARK ROYAL

The Park Royal area within the City of Mississauga, located adjacent to Metropolitan Toronto, has experienced basement flooding during heavy rainstorms in the past four years. This area is served by a separate storm sewer system.

After an extensive study, it was concluded that the flooding problems basically originated from the fact that the foundation drains for many of the homes in the area were connected to both storm and sanitary sewers, thus causing storm water to enter the sanitary system during times when the surcharge level would reach the level of the foundation drains. This in turn would cause the sanitary sewers to be overloaded, resulting in back-ups in basements.

To alleviate the problem, it would be necessary to either disconnect all foundation drains from the storm sewer system or apply the inlet control method. Since the first method would not be practical, we selected the latter.

Similar to the Bridlewood project, the recommended solution also incorporates regulators in selected catchbasins, with others sealed off. However, downspouts were left unrestricted.

Open spaces in the area were utilized for detention, by restricting storm sewer capacity downstream of the detention facilities and diverting the excess flows along with the road flows to the dry detention basins. A ten year level of protection was required by the City of Mississauga, but the system will accommodate higher intensity storms if a significant portion of the downspouts can be disconnected.

MODEL SELECTION

In order to establish the cause of the flooding, which was not readily apparent, we had to develop several hypotheses. Each of these was then checked against actual recorded flooding conditions. To prove the effect of foundation drains being connected to both storm and sanitary sewers, a model with surcharge capability was required. The HVM model was selected for the analysis of the system under existing conditions. Since the solution will result in elimination of surcharge, the Illudas model was used in the design of the relief works.

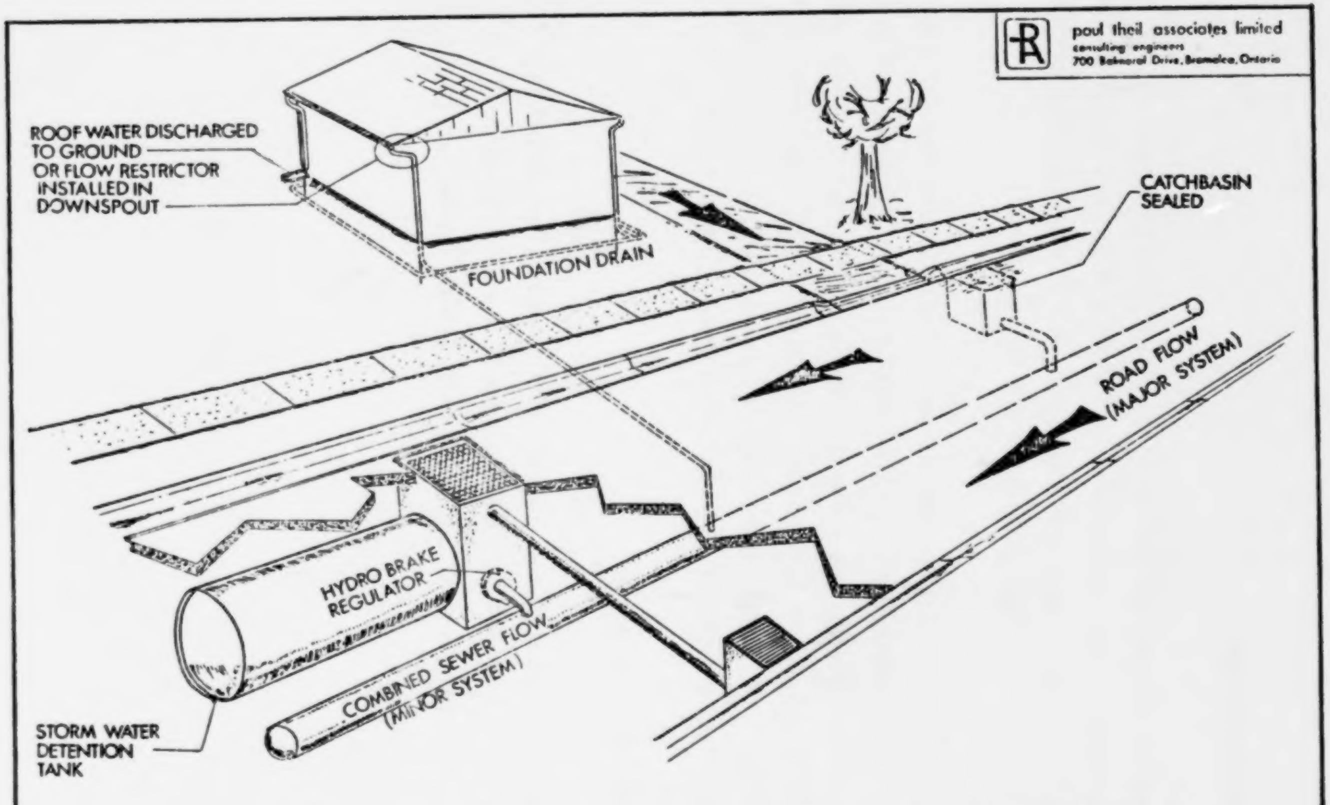
COMPUTER MODELLING

Modelling of the storm and sanitary sewer interconnection for the number of homes where such connections had been indicated, produced inflow rates into the sanitary sewer capable of surcharging the sanitary sewer to the levels that had been reported in the field.

The proposed flood relief works were modelled with the ILLUDAS program assuming the roadway and sewer flows to be one system. Once hydrographs had been established, then the allowable storm sewer flows, accounting for roof downspout contribution, were subtracted resulting in the expected road flows. The detention storage basins were modelled with the ILLUDAS modified storage option and storage volumes established.

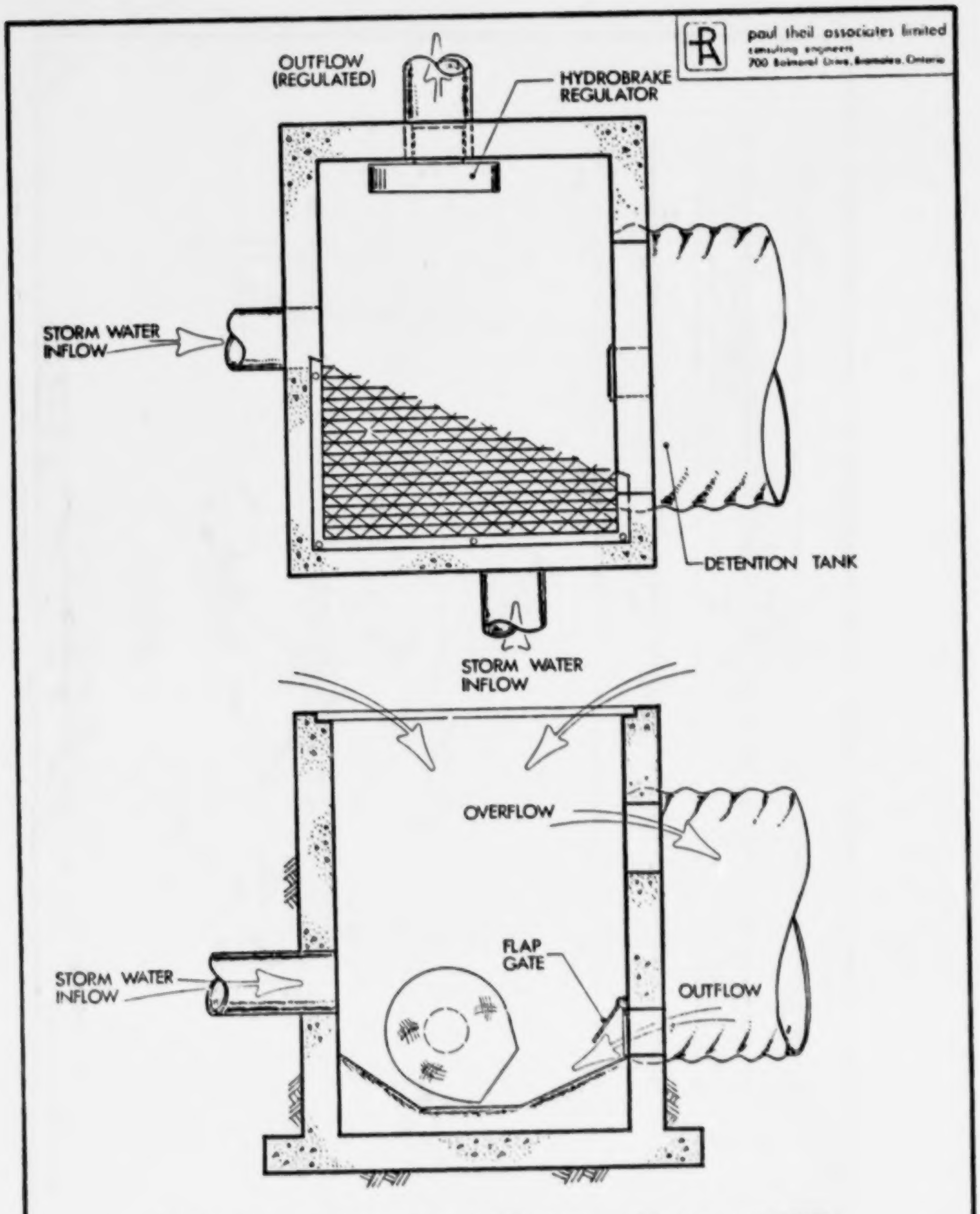
ESTIMATED COST OF WORKS

The cost of utilizing the inlet control method to solve the flooding problem in the Park Royal area is estimated to be about $350,000. This will provide protection for a 10 year design storm for about 1200 homes.


paul theil associates limited
consulting engineers
700 Balmoral Drive, Bramalea, Ontario
ROOF WATER DISCHARGED TO GROUND OR FLOW RESTRICTOR INSTALLED IN DOWNSPOUT
FOUNDATION DRAIN
CATCHBASIN SEALED
ROAD FLOW (MAJOR SYSTEM)
HYDRO BRAKE REGULATOR
COMBINED SEWER FLOW (MINOR SYSTEM)
STORM WATER DETENTION TANK

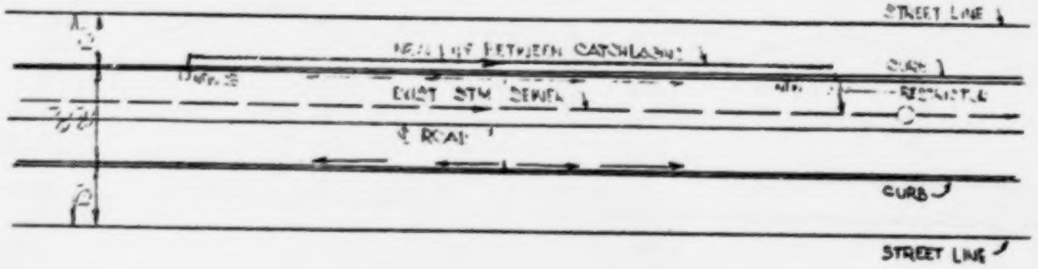
INLET CONTROL METHOD WITH DETENTION

FIG.1

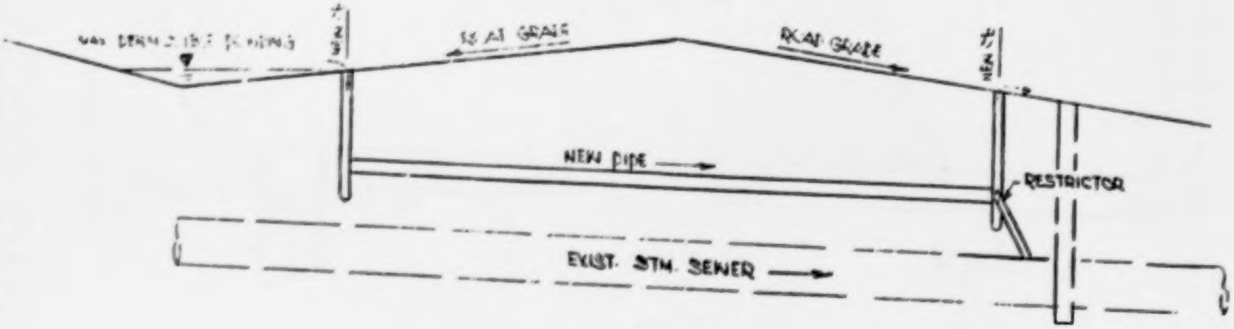

paul theil associates limited
consulting engineers
700 Balmoral Drive, Bramalea, Ontario
OUTFLOW (REGULATED)
HYDROBRAKE REGULATOR
STORM WATER INFLOW
DETENTION TANK
STORM WATER INFLOW
OVERFLOW
FLAP GATE
STORM WATER INFLOW
OUTFLOW

HEAD REGULATING CHAMBER

FIG 2


STREET LINE
NEW LINE BETWEEN CATCHBASINS
CURB
EXIST. STM. SEWER
RESTRICTOR
℄ ROAD
CURB
STREET LINE

PLAN


ROAD GRADE
NEW PIPE
RESTRICTOR
EXIST. STM. SEWER

PROFILE

INVERTED SIPHON

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FIG. 3

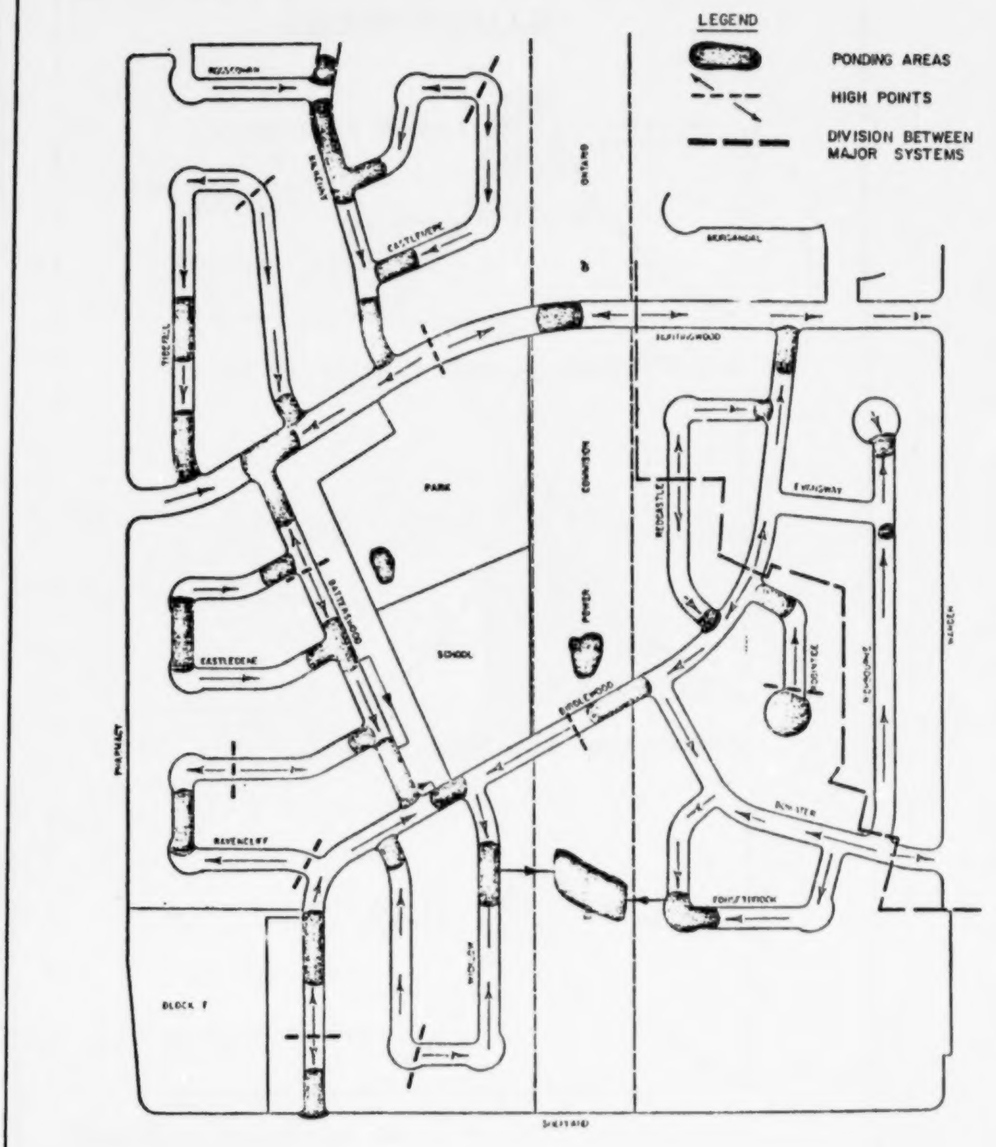
LEGEND
PONDING AREAS
HIGH POINTS
DIVISION BETWEEN MAJOR SYSTEMS
ROSSTOWN
TIDEFALL
CASTLEVIEW
ONTARIO
OF
COMMISSION
POWER
BERGANDAL
HUNTINGWOOD
PARK
SCHOOL
BATTERSWOOD
EASTLEDENE
RAVENCLIFF
PHARMACY
BLOCK F
BRIDLEWOOD
REDCASTLE
EVANSWAY
DUNSTER
FORESTBROOK
WARDEN
SHEPPARD

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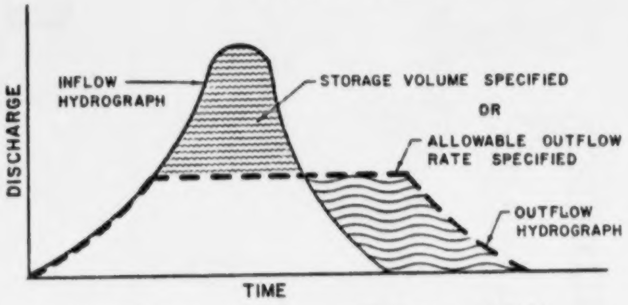
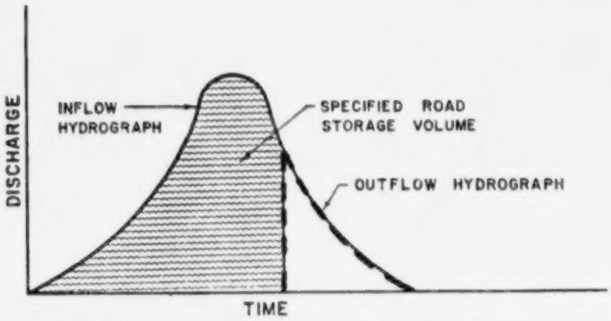
FIG. 4

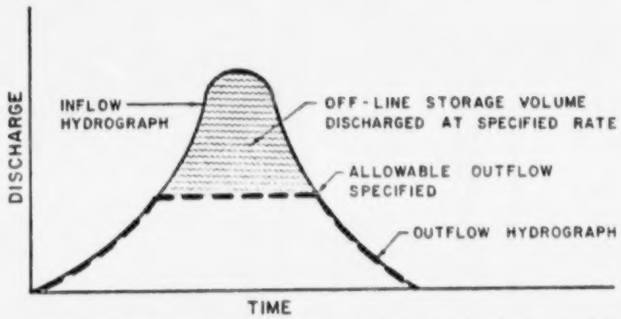
APPLICATION OF STORAGE OPTION IN THE ILLUDAS MODEL


INFLOW HYDROGRAPH
STORAGE VOLUME SPECIFIED
OR
ALLOWABLE OUTFLOW RATE SPECIFIED
OUTFLOW HYDROGRAPH
DISCHARGE
TIME

CONVENTIONAL ILLUDAS STORAGE OPTION


INFLOW HYDROGRAPH
SPECIFIED ROAD STORAGE VOLUME
OUTFLOW HYDROGRAPH
DISCHARGE
TIME

MODIFICATION FOR ROAD STORAGE


INFLOW HYDROGRAPH
OFF-LINE STORAGE VOLUME DISCHARGED AT SPECIFIED RATE
ALLOWABLE OUTFLOW SPECIFIED
OUTFLOW HYDROGRAPH
DISCHARGE
TIME

MODIFICATION FOR OFF-LINE DETENTION STORAGE

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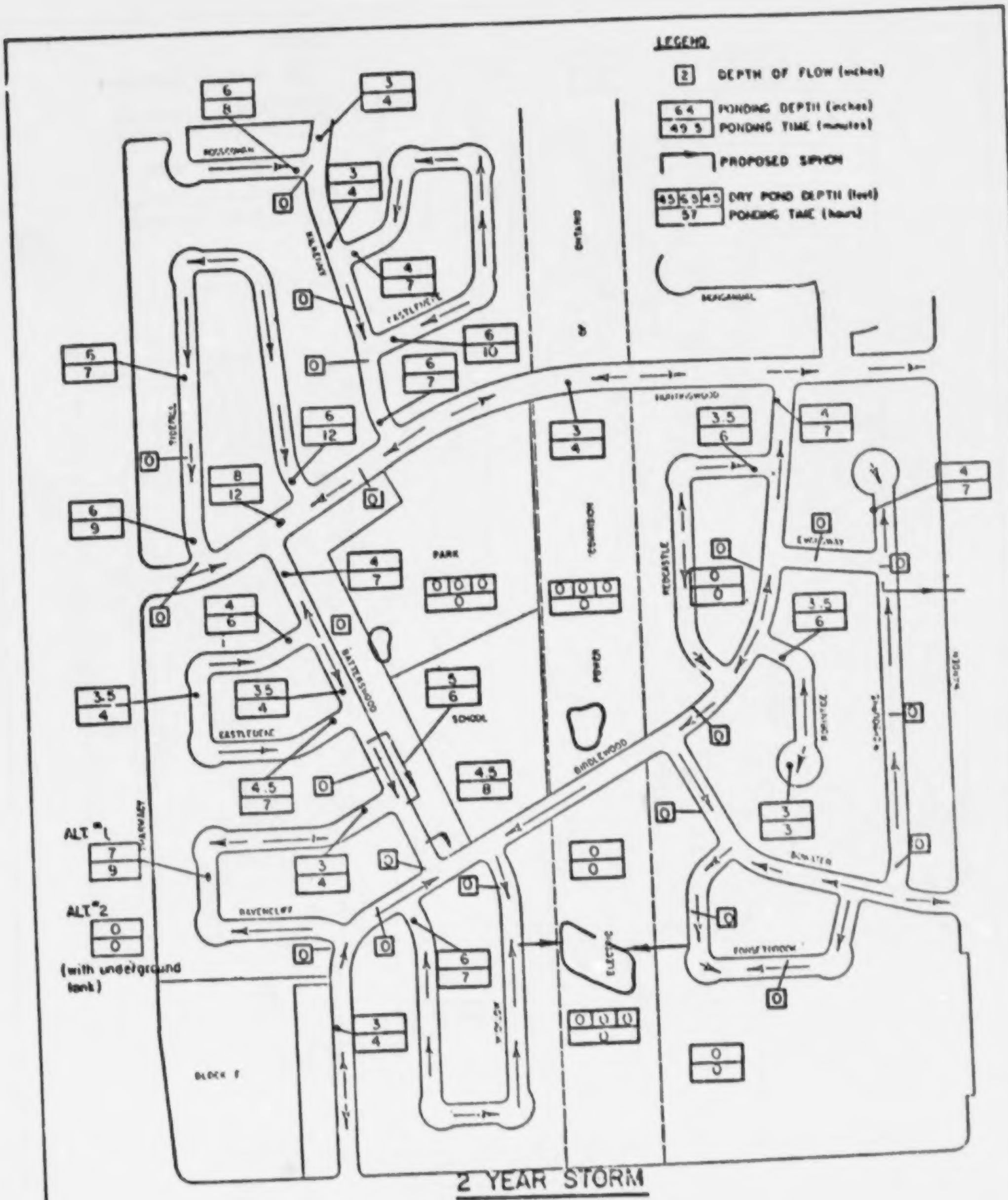
FIG.5


LEGEND
2 DEPTH OF FLOW (inches)
6.4 PONDING DEPTH (inches)
49.5 PONDING TIME (minutes)
PROPOSED SIPHON
4.5 5.5 4.5 DRY POND DEPTH (feet)
57 PONDING TIME (hours)
ONTARIO
HYDRO
POWER
PARK
SCHOOL
ELECTRIC
BATTLEWOOD
BIRDLEWOOD
CASTLEVIEW
CASTLEFIELD
RAVENCLIFF
REDCASTLE
BLOCK F
ALT. #1
7
9
ALT. #2
0
0
(with underground tank)

2 YEAR STORM

(ORIGINAL DESIGN)

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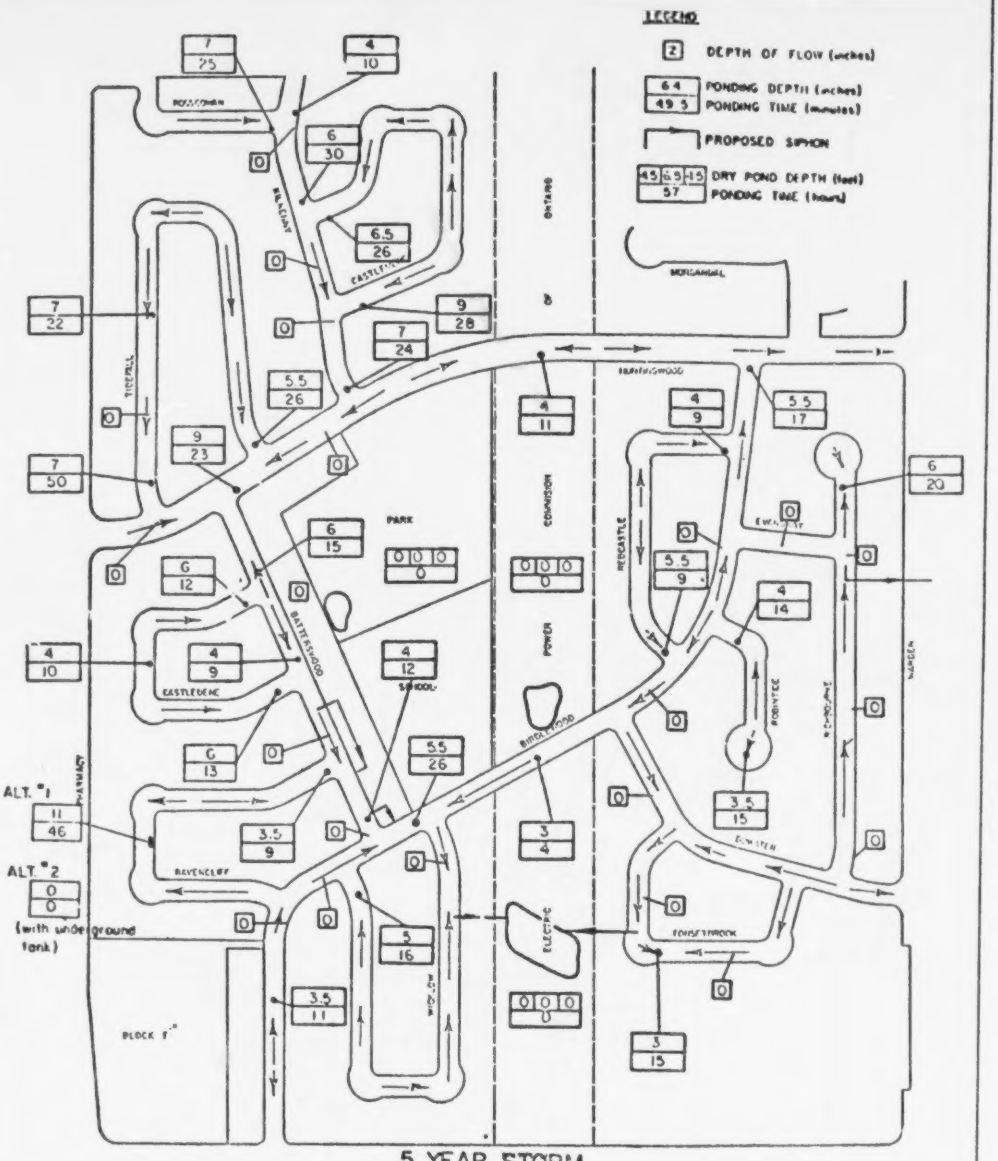
FIG. 6


LEGEND
2 DEPTH OF FLOW (inches)
6.4 PONDING DEPTH (inches)
49.5 PONDING TIME (minutes)
PROPOSED SIPHON
4.5 6.5 1.5 DRY POND DEPTH (feet)
57 PONDING TIME (hours)
ONTARIO
HYDRO
COMMISSION
POWER
ELECTRIC
PARK
SCHOOL
BATTERSWOOD
BRACHAY
CASTLEDENE
RAVENCLIFF
TIDEFALL
PHARMACY
MORGANDAL
REDCASTLE
ROBINTREE
BIRCHLEY
BLOCK "F"
ALT. #1
11
46
ALT. #2
0
0
(with underground tank)

5 YEAR STORM

(ORIGINAL DESIGN)

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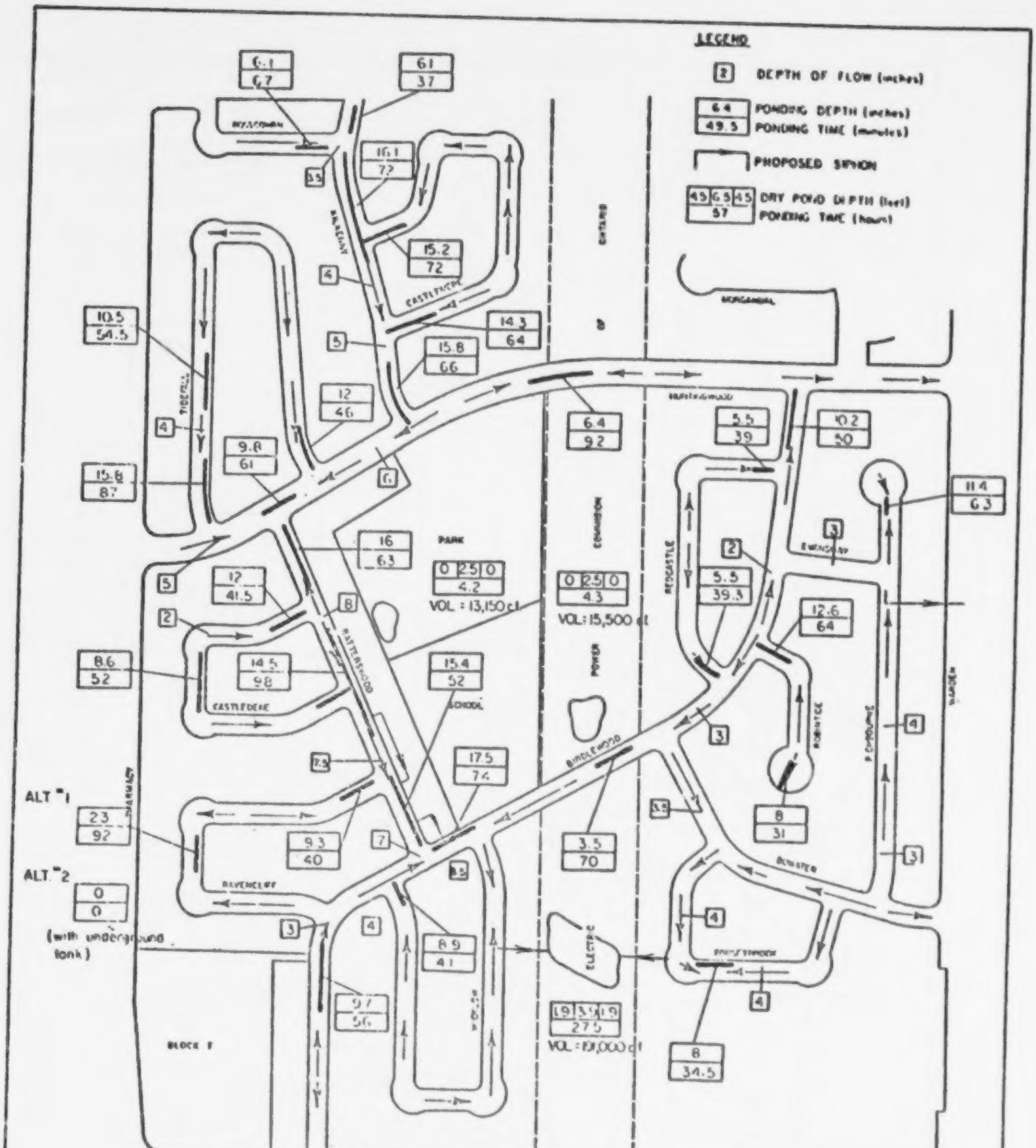
consulting engineers
700 Balmoral Drive Bramalea, Ontario

FIG. 7


LEGEND
2 DEPTH OF FLOW (inches)
6.4 PONDING DEPTH (inches)
49.5 PONDING TIME (minutes)
PROPOSED SIPHON
4.5 6.5 4.5 DRY POND DEPTH (feet)
57 PONDING TIME (hours)
6.1 / 6.7
61 / 37
16.1 / 72
15.2 / 72
14.3 / 64
15.8 / 66
10.5 / 54.5
12 / 46
9.8 / 61
15.8 / 87
6.4 / 92
5.5 / 39
10.2 / 50
11.4 / 63
16 / 63
PARK
17 / 41.5
0 2.5 0 / 4.2
VOL = 13,150 cf
0 2.5 0 / 4.3
VOL = 15,500 cf
5.5 / 39.3
12.6 / 64
8.6 / 52
14.5 / 98
15.4 / 52
SCHOOL
CASTLEDENE
CASTLEFROME
17.5 / 74
BIRDLEWOOD
ALT.#1
23 / 92
ALT.#2
0 / 0
(with underground tank)
9.3 / 40
RAVENCLIFF
3.5 / 70
8 / 31
8.9 / 41
9.7 / 56
1.9 3.9 1.9 / 27.5
VOL = 19,000 cf
8 / 34.5
BLOCK F
ELECTRIC
POWER
COMMISSION
ONTARIO
HYDRO
REDCASTLE
ROBINTREE
3.5
2.5
7
8
6
5
4
3
2

25 YEAR STORM

(ORIGINAL DESIGN)

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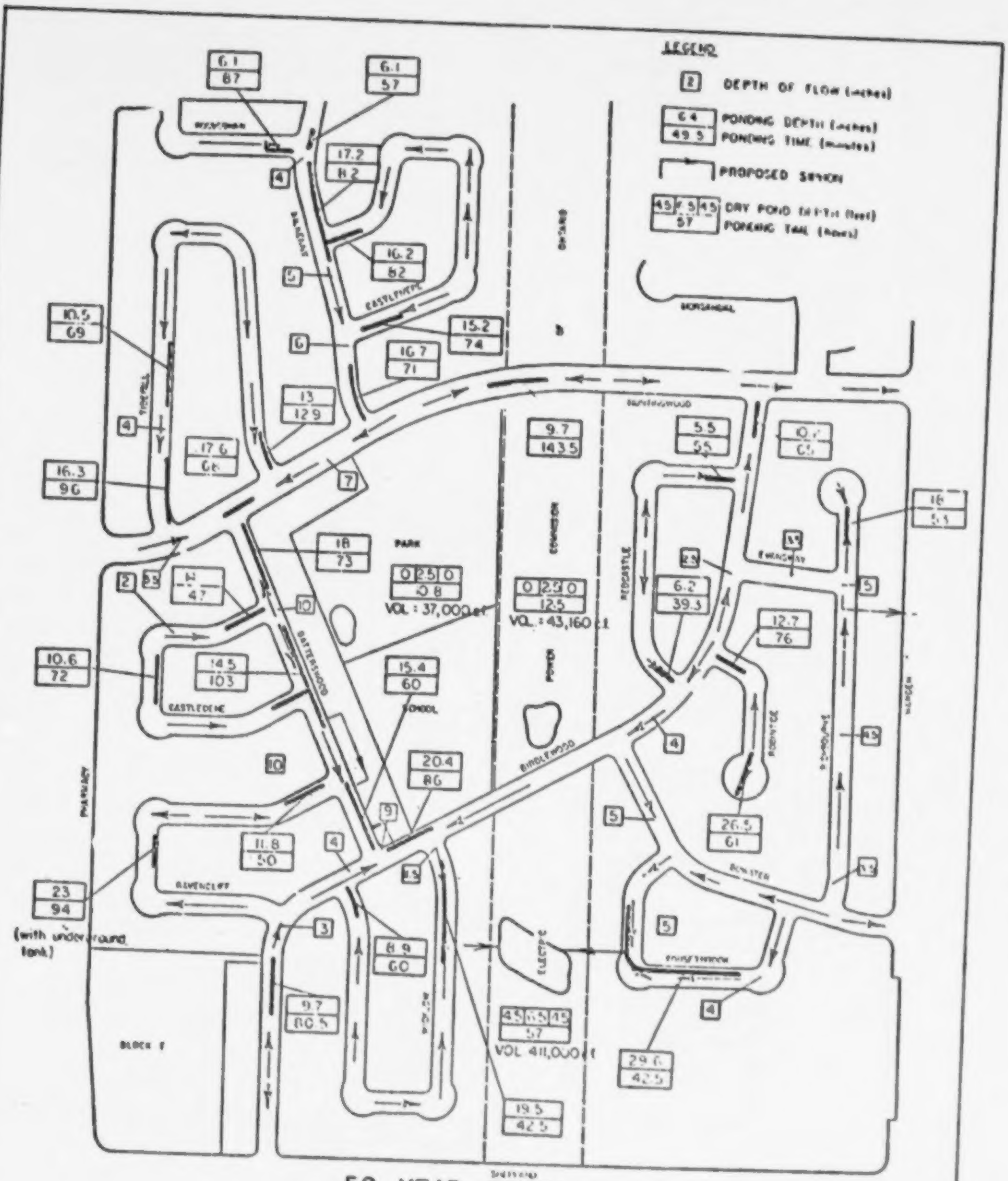
consulting engineers
700 Balmoral Drive, Bramalea, Ontario

FIG. 8


LEGEND
2 DEPTH OF FLOW (inches)
6.4 / 49.5 PONDING DEPTH (inches) PONDING TIME (minutes)
PROPOSED SWALE
4.5 7.5 4.5 / 57 DRY POND DEPTH (feet) PONDING TIME (hours)
6.1 / 87
6.1 / 57
17.2 / 112
16.2 / 82
15.2 / 74
16.7 / 71
13 / 129
10.5 / 69
17.6 / 66
16.3 / 96
18 / 73
0 2.5 0 / 0.8
VOL : 37,000 c.f.
0 2.9 0 / 12.5
VOL : 43,160 c.f.
9.7 / 143.5
5.5 / 55
16 / 53
6.2 / 39.5
12.7 / 76
14.5 / 103
10.6 / 72
15.4 / 60
20.4 / 86
11.8 / 50
23 / 94
(with underground tank)
9.7 / 80.5
8.9 / 60
4.5 6.5 4.5 / 57
VOL 411,000 c.f.
19.5 / 42.5
26.5 / 61
29.6 / 42.5
PARK
SCHOOL
POWER CORRIDOR
ELECTRIC
BATTERSWOOD
CASTLEDENE
RAVENCLIFF
BIRDLEWOOD
REDCASTLE
BLOCK F
PARKWAY

50 YEAR STORM

(ORIGINAL DESIGN)

paul theil associates limited
consulting engineers
700 Balmoral Drive, Bramalea Ontario

FIG. 9

ANALYSIS OF DETENTION BASIN SYSTEMS

by

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ANALYSIS OF DETENTION BASIN SYSTEMS

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INTRODUCTION

In recent years there has been an increased interest in the use of detention basins as a means of reducing peak flows on rivers and streams. This interest is focused particularily on urban or urbanizing catchments where the increased volumes and peaks of flood flows, combined with the practice of building in the flood plain, can create unacceptable risks to both life and property.

The most common type of basin consists of an embankment placed across the stream with an uncontrolled conduit outlet through the bottom. The temporary storage thus created has the effect of both attenuating and delaying the peak of the flood hydrographs passing through it. An overflow spillway set near the top of the embankment is required to safely discharge those large floods which would otherwise exceed the basin capacity.

Despite the reduction in peak flows due to the construction of a detention basin it does not inevitably follow that flood damages and levels will be reduced downstream. It sometimes happens that false confidence in the protection given by a basin gives rise to increased development in the region downstream. Consequently, for floods larger than the basin design flood, damage caused far exceeds that which would have occured had the basin not been built. In some cases the result is that average annual damages are in fact increased. Another possibility is that the delay of the flood peak which results from routing the hydrograph through the basin storage can lead to an undesirable interaction with downstream tributrary hydrographs (McCuen, 1975; Morris, 1976).

This paper briefly reviews the method of design of detention basins and describes a model suitable for the analysis and simulation of multiple basin systems. An example application to an urban system is given, although the method and concepts apply equally to rural catchments.

ESSENTIAL CONCEPTS

The main features of a detention basin are shown in Figure 1. The basin storage is created by placing an embankment across the stream. The level of the outlet conduit can be placed so as to provide permanent storage, or left at the level of the basin floor so that the basin is usually dry; multiple uses of basins such as for recreation and/or wildlife purposes are common.

A typical stage-discharge curve for a basin is shown in Figure 2. The initial portion of the curve is for weir flow over the horizontal pipe inlet changing at some value of stage into full pipe flow. (The transition point is difficult to predict, and sometimes special devices are used to induce pipe flow to occur early.) The pipe flow characteristic in which the discharge varies only slowly with change in basin water level is a feature of detention basins and produces a flat-topped outflow hydrograph as shown in Figure 3. For the basin design flood the basin should just fill to the level of the overflow spillway (Figure 1).

For floods larger than the design flood part of the outflow is passed over the spillway. The essential characteristic of this spillway as shown in Figure 2 is the rapid increase in discharge for small increase in stage. A standard overflow spillway, or a glory hole spillway operating in the weir flow range, is suitable. Figure 3 shows the rapid changes in discharge caused when floods exceed the design basin capacity--these rapid changes can be a hazard downstream and should be minimized as much as possible by limiting the effective spillway length.

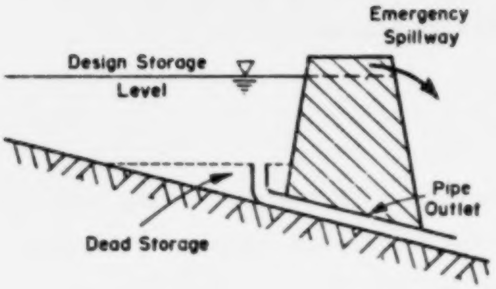
Emergency
Spillway
Design Storage
Level
Pipe
Outlet
Dead Storage

Figure 1 Schematic diagram of a typical detention basin showing essential features.

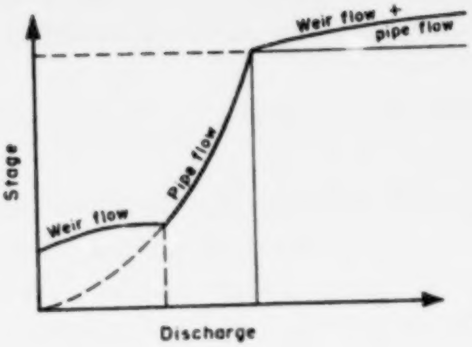

Figure 2 Stage-discharge curve for a detention basin with features as in Figure 1.

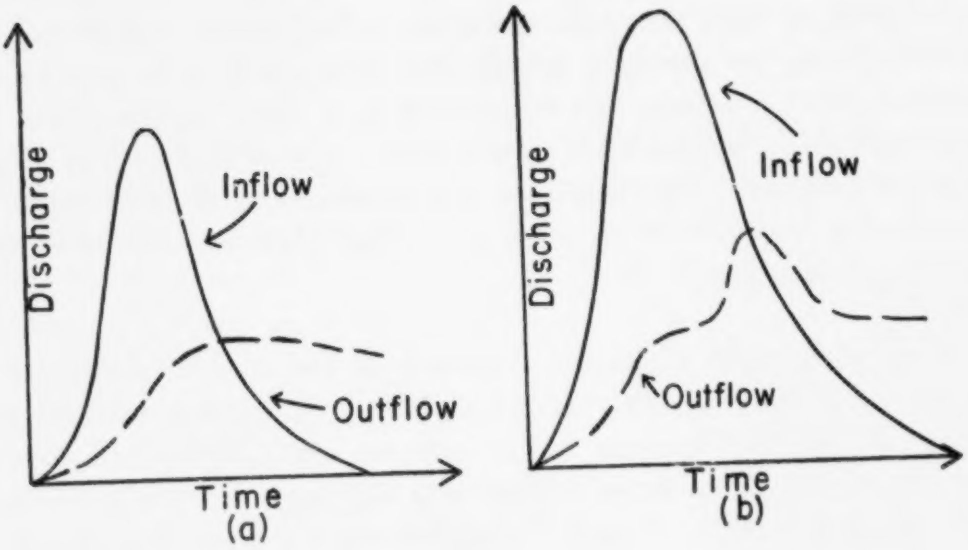

Figure 3 Typical inflow and outflow hydrographs for a detention basin. (a) for design storm (b) for extreme storm.

Three design variables interact with one another in determining basin dimensions:

(i) recurrence interval of design flood,

(ii) peak outflow discharge through the pipe (which occurs when basin is full), and

(iii) basin storage capacity.

Once any two of these are chosen, the third one is automatically fixed. Thus for a given basin size and design flood recurrence interval, the percent reduction in peak flow possible (and therefore the outlet capacity) is fixed by the requirement that the basin just fill during the passage of the design flood.

The term "recurrence interval" is used in this paper in connection with flood peak flows (or levels) downstream of the basin site. For example, a 20 year recurrence interval flow is that flow which is equalled or exceeded on the average once every 20 years. If no basin is present this definition presents no problems. If, however, a detention basin is introduced to control floods up to a 20 year recurrence interval it means that, on the average, the basin will fill or overflow (over the spillway) once every 20 years and all smaller floods will pass through the pipe outlet. The inflow flood corresponding to the basin-just-filling condition is not the same flood which caused 20 year peak flows prior to its construction; it will have come from a longer duration storm, have more volume, and a lower peak. That is, the basin inflow flood need not necessarily have a 20 year peak nor a 20 year flood volume; it is the peak outflow which is the important parameter.

BASIN DESIGN PROCEDURE

The design procedure for a basin, whether it be a basin in isolation or one basin of many in a system, involves several trial calculations. This is because the duration of the critical design storm for the chosen design recurrence interval is not known in advance; it is required to route design hydrographs for a range of durations through the basin and to select the "worst" case.

The design calculations are usually for either (i) the basin capacity required to achieve a given peak flow downstream; here, the design variables are the overflow spillway level and the diameter and/or number of outlet pipes, or (ii) the outlet size for a basin whose capacity is fixed (by topography); here the design variables are the diameter and/or number of outlet pipes. In both of these cases an iterative procedure is necessary.

A similar procedure is required for the sizing of the overflow spillway. Extreme floods of various durations are routed through the basin for various spillway sizes to determine some optimum combination of spillway cost vs. embankment cost for critical storms. The required embankment height is then determined by adding freeboard to the peak water level reached.

Finally the effect of each basin on the flow peaks downstream should be investigated. This will determine not only the effectiveness of the proposed basin but also whether any adverse effects would be caused by its construction.

Table 1 summarizes the steps required in the final design procedure.

Table 1.SUMMARY OF BASIN DESIGN PROCEDURE.

Basin capacity and outlet sizes to be determined such that allowable peak flow downstream is not exceeded.	For capacity fixed. Design to size outlet works.
(i) Compute design inflow hydrograph for a range of storm durations for the required recurrence interval.	(i) Compute design inflow hydrographs for a range of storm durations for the required recurrence interval.
(ii) For each storm in (i) adjust the diameter and/or number of outlet pipes such that allowable peak flow downstream is not exceeded. Note the maximum water level and basin capacity required.	(ii) For each storm in (i) adjust the diameter and/or number of outlet pipes until the basin just fills during the passage of the flood hydrograph.

Table 1. SUMMARY OF BASIN DESIGN PROCEDURE (Continued)

Basin capacity and outlet sizes to be determined such that allowable peak flow downstream is not exceeded.	For capacity fixed. Design to size outlet works.
(iii) Design basin capacity is the largest required in (ii). This gives the level of the overflow spillway. Size of outlet works is obtained from the same routing calculation that gave the design capacity.	(iii) Size of outflow works is obtained from worst case in (ii). (i.e. the largest capacity outflow required)
(iv) Design overflow spillway dimensions determined by routing extreme floods for a range of durations through the basin designed in (iii) and selecting "worst" case.	(iv) Design overflow spillway dimensions determined by routing extreme floods for a range of storm durations through the basin designed in (iii) and selecting "worst" case.
(v) Check effects of basin on flows downstream.	(v) Check effects of basin on flows downstream.

BASIN DESIGN METHODS

The final design of basins linked in series and/or parallel in a multi-basin systems requires the computation of inflow hydrographs at many points, the routing of hydrographs through the basins, and the routing of the outflow hydrographs downstream (with addition of tributrary hydrographs where appropriate).

A suitable mathematical catchment model which allows for temporal and areal variation of rainfall, for areal variation of losses, and for calculation of hydrographs at any point on a catchment is based on the development Laurenson model (Laurenson, 1964) described for rural catchments by Mein et al.(1974). In this model the catchment is divided into subcatchments along watershed lines; the rainfall-excess for each is computed from the temporal pattern of the nearest recording rain gage and the depth as found from the isohyetal map of the storm event. Catchment storage effects are simulated by routing flows between subcatchments through simple nonlinear storage elements. Determination of the parameters for these storages is discussed elsewhere (Mein et al., 1974; Laurenson and Mein, 1978).

Subsequent work toward the application of the model to urban areas (Mein and Woodhouse, 1977; Crouch and Mein, 1978) has led to the current version of the model (Laurenson and Mein, 1978) which is being increasingly used by the engineering profession in Australia. It is a general model which is suitable for the estimation of design flood hydrographs from storm data for rural, urban, or partly urban catchments. A detention basin design subroutine is also included and the program runs interactively to allow the user to design the basin system on-line.

EXAMPLE APPLICATION OF MODEL

Gardiners Creek is situated in the eastern suburbs of Melbourne, Australia, approximately 12 km east of the city center. Just below the junction with Scotchmans Creek the catchment area is 64 km^2 and is fully urbanized, being mainly residential. As can be seen in Figure 4 there are ten detention basins on the catchment, some in series with one another, others in parallel. One, Lake Road, uses the upper portion of Blackburn Lake as the detention basin; the others are normally dry. All were constructed in the period 1963-1968. Storage sizes vary from 0.05-1.0 (10^6 m^3).

Rainfall is recorded by 15 to 30 standard rainguages and by five pluviometers throughout the catchment area. Runoff is not measured directly, but the stage above some dry weather flow level is recorded on charts at each basin. Rating curves, based on theoretical equations for flow through the conduits and over the overflow spillways, were provided by the Melbourne and Metropolitan Board of Works.

After inspection of the rainfall and runoff records three storm events were chosen for analysis. The May 1974 storm was used for fitting the model; two others (September 1975, December 1975) were reserved for testing it.

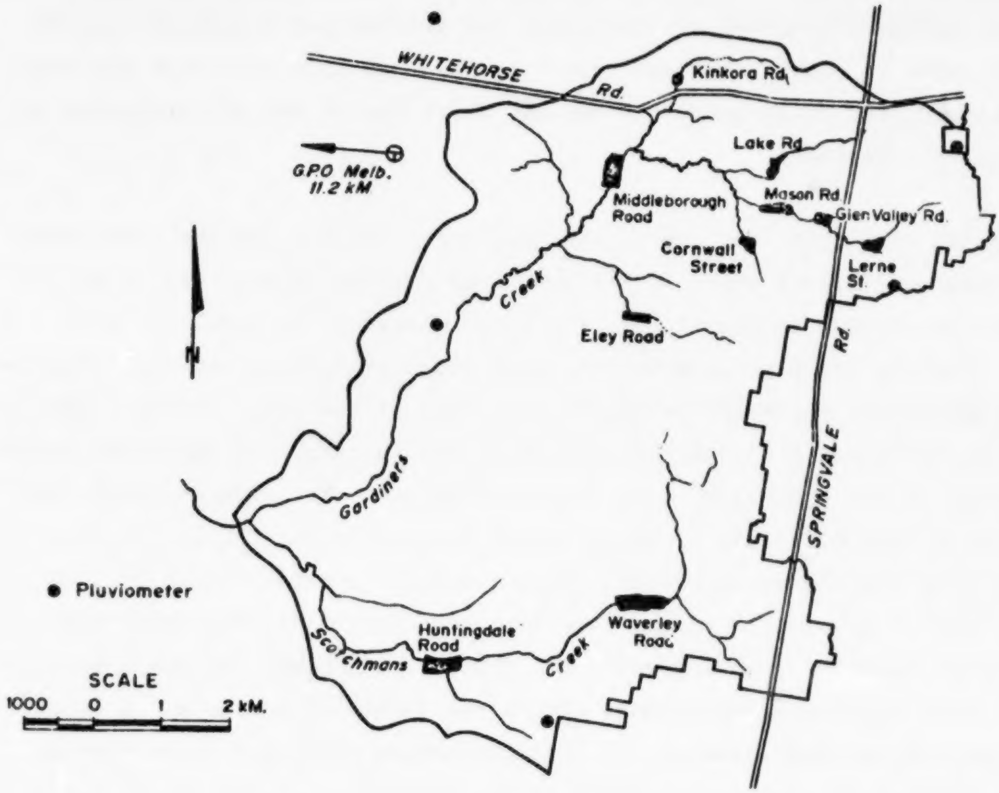
WHITEHORSE Rd.
Kinkora Rd.
Lake Rd.
G.P.O. Melb.
11.2 kM
Middleborough Road
Mason Rd.
Glen Valley Rd.
Cornwall Street
Lerne St.
N
Creek
Eley Road
Rd.
SPRINGVALE
Gardiners
● Pluviometer
Scotchmans
Huntingdale Road
Creek
Waverley Road
SCALE
1000
0
1
2 kM.

Figure 4 Gardiners Creek catchment in Melbourne (Australia) showing location of detention basins.

Following processing of the charts and rainfall data, the hydrograph corresponding to the stage-hydrograph at each basin was able to be deduced using the theoretical rating curves. It was then possible to compute the implied losses for the catchment area of each basin by subtracting the runoff from the rainfall. Some anomalies immediately became apparent; blockage, or partial blockage, of at least two basin outlets for some events meant that the derived outflow hydrograph was incorrect and the deduced volume of outflow was too high. A slower than expected transition to pipe flow also affected the rating curves in at least one basin. There was no way of correcting the hydrographs concerned.

The results of the fit and one of the test runs (results for the other test storm were similar)are shown in Figures 5 and 6 respectively. It is not proposed to discuss in detail here the performance of the model for each basin. Some of the poor agreement between model prediction and the "observed" hydrograph can be explained by partial blockage of the basin outlet. The Lerne St. basin was affected for each storm and in turn influenced the inflow hydrograph to the Glenvalley Road basin downstream. The Middleborough Road basin outlet was partially clogged with debris during the May 1974 storm. The Waverley Road basin developed a vortex which prevented full pipe flow for at least the May 1974 storm; this in turn affected the inflow to the Huntingdale Road basin downstream. All things considered, the model was judged to be performing acceptably within the limits of the data. A point worth keeping in mind, however, is that the basins provide a large damping effect which could mask small errors in the generation of the inflow hydrographs.

Once the model has been fitted and tested it can be used to examine the effect of the detention basins on the catchment outflow hydrograph for any storm. For example, for the May 1974 storm the basins reduced the peak flow at the catchment outlet by 18%. Within the catchment the peak reductions achieved were variable and as high as 70%. It would also be a simple matter to examine the effect of each basin on flows at any particular point for any storm. Thus, once the model is fitted and tested, it is a powerful tool for

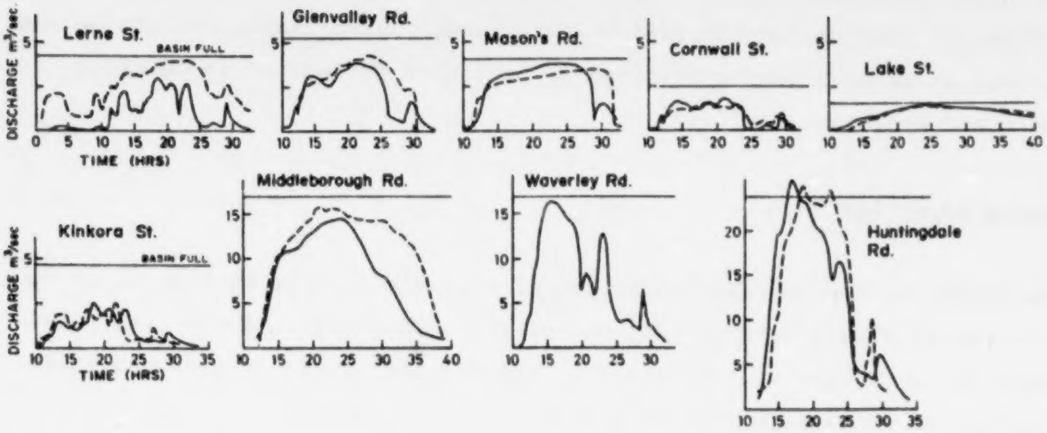

Figure 5 Performance of the model after fitting for May 1974 storm. Recorded hydrographs are shown by dashed lines.

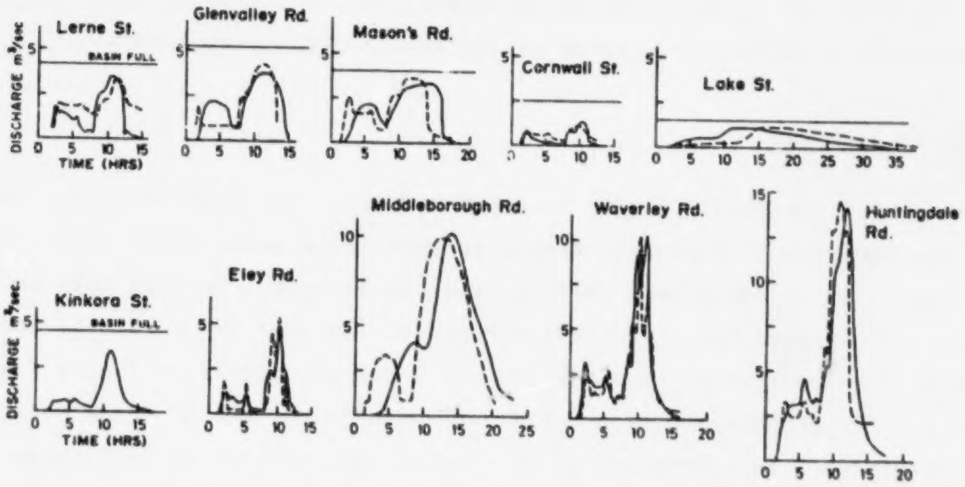

Figure 6 Results for test storm (September 1975). Dashed lines are recorded hydrographs.

for analysis of the basin system.

GENERALIZED RESULTS

The model described above was developed for application to any particular catchment. Having demonstrated that the model is suitable, it can be used to look at general characteristics of basins in series and in parallel (Mein and Woodhouse, 1977).

Basins in series

The effect on the outflow hydrograph of a single storage of size S, of two storages of size S/2 in series, and of three size S/3 in series, were compared for a hypothetical catchment. The results generally agreed with the theoretical analysis of Wycoff and Singh (1976) and show:

(i) that for a downstream point a single basin reduces the peak discharge by more than the same storage quantity distributed over two basins in series, and that two basins are more effective than three. Multiple basin systems can of course, reduce the peaks over intermediate stream sections;

(ii) the effect of a detention basin is greatest at its outlet, and diminishes fairly rapidly downstream as additional area contributes.

Basins in parallel

McCuen (1974) and Morris (1976) have both reported calculations which showed that the addition of detention basins in particular cases would have increased the peak flows in downstream reaches by a few percent. Perhaps this problem is best illustrated by a hypothetical example. Consider two confluent streams on which the hydrographs from a storm are shown separately in Figure 7 as A and B. The constuction of a detention basin on one stream could modify the hydrograph A to hydrograph A' with a reduced peak but with an extended tail. The combination of A' and B then would give a higher peak flow at the stream junction than did the combination of A and B. Thus the detention basin has increased the peak flows downstream of the junction. [It is worth

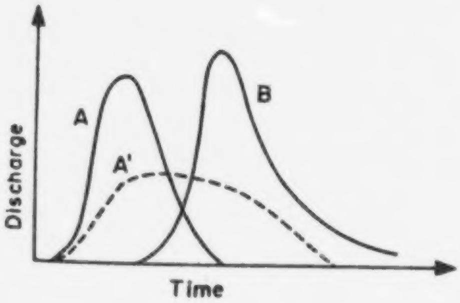

Figure 7. Showing how a detention basin (output A') can attenuate and delay an inlet hydrograph A so that it now interacts with hydrograph B on a downstream junction.

noting here that catchments which display separated response hydrographs as depicted by A and B are very uncommon].

Many configurations of single basins and of basins in parallel were tried in the model in an attempt to define the conditions under which the downstream flow peaks could be increased. In only one case did an increase result, and even then it was only by 2%.

It was concluded from these studies that it is certainly unusual for the addition of a basin to increase the flood peaks downstream. It is nevertheless possible that a particular basin or combination of basins could have this effect; thus it is important that multibasin systems be fully analysed to determine the individual contribution (or otherwise) of each basin to flood mitigation. With the model described above these checks can easily be made.

RAPID PRELIMINARY PROCEDURES

Table 1 shows that a large number of routings for various combinations of storm duration are necessary for basin design at each potential site. For preliminary studies of single basin systems a number of simple methods have been proposed.

Culp (1948) proposed the formula:

$$(Qo/Qi) = 1.25 - \sqrt{1.5(Vs/Vi) + 0.063}$$

where Qo = peak outflow rate
Qi = peak inflow rate (same units as Qo)
Vs = volume of storage of basin
Vi = volume of inflow hydrograph (same units as Vs)

This formula is applicable for basins with a conduit outlet operating in the pipe flow range and for which a triangular inflow hydrograph is a satisfactory approximation. It also shows quite clearly the fixed relationship between recurrence interval of the design flood (given by Qi and Vi), the basin volume (Vs), and the peak outflow discharge (Qi), as discussed above. In the author's experience it gives quite good estimates of basin capacity.

Gould (1968) published a formula based on similar assumptions concerning the inflow hydrograph but extended the analysis to include the duration of the design storm. The results were presented in chart form to enable rapid estimates to be made.

For basins on small catchments rapid procedures may be adequate for final design. Poertner (1974) recommends use of the Rational formula to compute inflow hydrographs for basins less than 20 acres (8 ha) in area; a simple outflow assumption enables the storage requirement to be quickly determined. For larger catchments however, the Rational method is not satisfactory to compute hydrographs for basin design.

For multiple-basin systems the problem is much less suited to simplifying assumptions. Attempts have been made (Wycoff and Singh, 1976; Abt and Grigg, 1978) but have been limited to the case of detention reservoirs in series. It is probable that a final design model which calculates and routes hydrographs is necessary for most multiple-basin analysis.

CONCLUSION

Design of, and investigation of the effectiveness of, detention basin systems is a complicated procedure not amenable to analysis by simple procedures (except perhaps for single basins on small catchments). A distributed model, such as the one used in this paper, is necessary to compute and route the hydrographs through the system. With such a tool the examination of the influence of each basin, or group of basins, on flood flows at any point is readily performed.

Few generalizations about systems of detention basins can be made. Just two remarks will be made here based on the use of the model:

(i) A single storage is more effective in reducing peak flows at a point than is a series of basins with the same combined storage capacity. The effect of a basin on peak reduction diminishes downstream as more area contributes.

(ii) It is unusual for a basin or for basins in parallel to cause higher flow peaks at a point downstream. Nevertheless, each particular case should be investigated for this effect using a variety of storms.

ACKNOWLEDGEMENTS

Application of the model to the Gardiners Creek catchment was done by Mr. M. P. Woodhouse. Data for this catchment were supplied by the Melbourne and Metropolitan Board of Works.

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CSO FACILITIES PLANNING IN CINCINNATI USING SWMM
(A CASE STUDY)

by Jeffrey D. Sharon, Project Engineer
Havens and Emerson, Inc., Cleveland, Ohio

INTRODUCTION

The U.S. EPA's 201 Facilities Planning criteria for combined sewer systems require a detailed analysis of overflow volumes and pollutant loads into the receiving streams. This paper presents a case history of excerpts of a facility planning effort where the EPA SWMM was used as a primary tool for examining the system of combined sewers in Cincinnati. The facilities plan was prepared by Havens and Emerson, Inc. for the Cincinnati Metropolitan Sewer District. The first discussion presented in the paper is the calibration of the SWMM, which includes a sensitivity analysis and plots of SWMM-simulated hydrographs versus observed field data for three distincly different test catchments. Next, the method used for simulating the hydraulics of the system's various regulators and overflows is briefly discussed. And finally, a description of the estimation of overflow pollutant loads into the receiving streams is presented.

STUDY AREA

The combined sewer system within the study area drains approximately 60,000 acres and serves nearly 500,000 people. The system discharges to three major watercourses (Mill Creek, Little Miami River, Ohio River) and some smaller tributaries through nearly 200 overflows. Numerous types of overflows exist ranging from float operated gates to simple diversion structures with or without weirs. Intercepted flow is conveyed to either of two major wastewater treatment plants which discharge effluent into the Ohio River.

CALIBRATION OF THE SWMM

Before setting up the model and conducting detailed simulations on the catchments in the study area, the model had to be calibrated using a comparison of field gagings with model output. Field measurements were made on several test catchments chosen to represent the study area. This included measurements of discharge versus time at the outlets to the chosen test basins as well as measurements of accumulating rainfall depths versus time at raingages located in or near each test basin. Runoff hydrographs and rainfall hyetographs were the results of plots of these data.

The model was initially run using the measured hyetographs along with a first attempt at input parameters (default values were used whenever possible). Various input parameters were then adjusted until the predicted results corresponded closely with the field gagings. Only after the model-predicted results corresponded closely with field data for various types of catchments under varying rainfall conditions was the model deemed to be verified sufficiently for extrapolated use over the entire study area.

The first step in the calibration procedure was the sensitivity analysis. The sensitivity analysis is a study of the model's response to the variance of certain parameters which are used as input. The primary purpose of the sensitivity analysis is to permit the user to become better acquainted with the model and establish the parameters to which the prediction of the runoff phenomena is most sensitive. This is particularly important when certain data are unobtainable or unreliable since it assists the user in making a decision as to whether he may proceed with simulation or must obtain more precise data. A successful sensitivity analysis leads to the calibration of the model and its subsequent use for simulation.

The sensitivity analysis for this study was conducted to confirm the results obtained in several previous analyses of the SWMM program and to ensure that such results applied to the ongoing study. Three previous studies (1), (2), (3) have concluded concurrently that the percent imperviousness of a catchment is the most sensitive variable in the process of calibration. Other parameters were found to be ranked in a somewhat different order from one report to the next.

Graham et al(1) ranked the parameters most affecting the program in descending order as follows: watershed imperviousness, pervious area infiltration rates, watershed length to width ratio, overland flow slope, pervious area coefficient of roughness, impervious area detention depths. Other parameters were found to have a negligible effect.

Wisner et al(2) stated that, "When studying the sensitivity of the SWMM - simulated runoff peak flows, the imperviousness was found to have the greatest impact, followed by the catchment width. The testing of the SWMM on several Canadian watersheds indicated that for the remaining runoff quantity parameters, the default values stored in the model may be used."

The Ontario Ministry of the Environment's Research Report No. 47(3) concluded similarly: "The percentage of imperviousness had the most significant effect on the surface runoff of all the parameters tested. Both the peak flow and volume of runoff were almost directly proportional to the amount of impervious area. The remaining parameters were ranked in order of their decreasing influence: width of overland flow, infiltration rates, retention depths, roughness coefficients and ground slopes. The SWMM default values for these parameters ...... lie on the relatively flat portions of the sensitivity curves, and appear to be reasonable estimates when no actual site measurements exist."

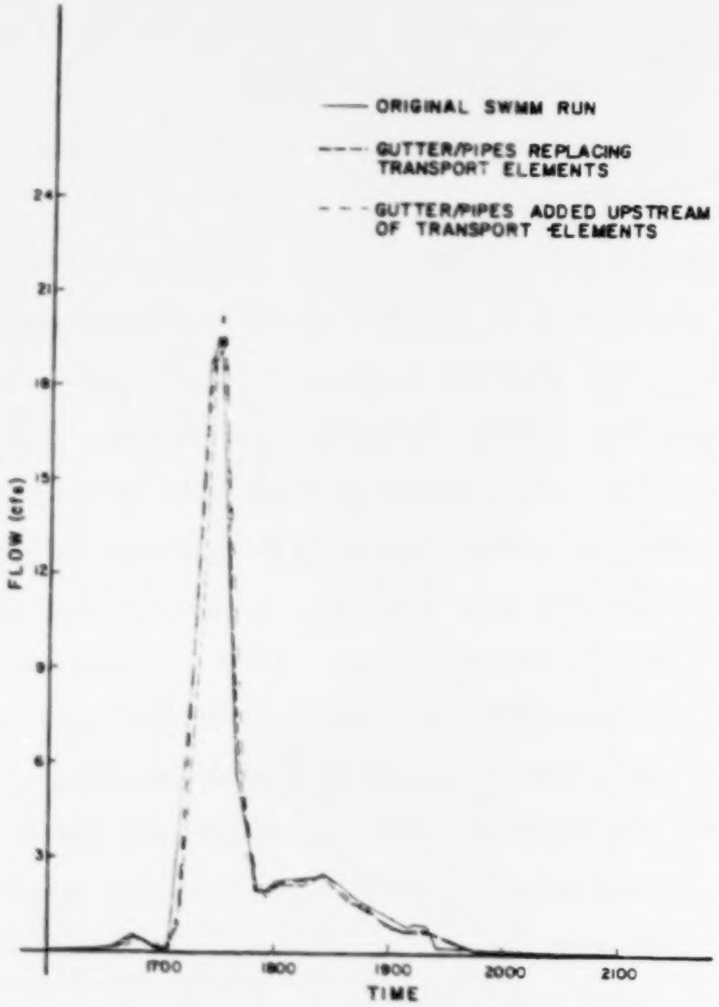
Most of the parameters that can be changed for the sensitivity analysis are utilized in the Runoff Block. However, a successful calibration must also include a look at Transport Block as well in order to ensure an accurate portrayal of routing delays through the sewer system as flows from numerous subcatchments are combined. Consequently, catchments of 3 distinct sizes were analyzed in the calibration procedures. These include the Grasselli (42.2 acres), Sunset (562 acres), and Bloody Run (2,347 acres) catchments.

It was the Grasselli Catchment which was used for most of the detailed sensitivity analysis. Grasselli, a typical residential catchment of uniform land use, was chosen since it could be subdivided into perfectly rectangular subcatchments with relatively uniform slope conditions exhibited by each. This ensured that the characteristic width and weighted slope calculations would be calculated using the precise methodology outlined in the User's Manual(4) and the Short Course Materials(5) without any estimations. Thus, two of the calibrated parameters were accurately fixed, thereby simplifying the original sensitivity analysis. Grasselli is also a small watershed which made computer runs relatively short and inexpensive for the detailed analysis.

Figures 1 through 6 present a synopsis of the sensitivity analysis performed on Grasselli. The July 15, 1976 rainfall was chosen for this study since it was a rainfall of relatively short duration (minimizing computer expense) and field measured runoff rates corresponded closely with the model-predicted flows.

Figure 1 addresses the variability of flow prediction with different discretization schemes. In the original SWMM run, each subcatchment discharged directly into Transport Block sewer elements through which the routing takes place. The second run replaced Transport sewer elements with Runoff Block gutter/pipe elements as receptacles for subcatchment overland flow runoff. No appreciable change in routing occurred. In the third run, the original Transport Block sewer elements were again utilized, this time in conjunction with Runoff Block gutter/pipes added upstream to receive subcatchment flows producing a much more discrete simulation. Again, no appreciable change in the hydrograph occurred. From this, it may be concluded that either the Transport elements or Runoff pipes may be used with the same degree of accuracy and that further discretization by adding gutter/pipes upstream of Transport elements is unnecessary. This may not be true, however, for more complicated catchments.

Figure 2 displays the effects of changing ground surface infiltration rates. Here, infiltration rates were doubled from a maximum and minimum infiltration rate of 3.00 inches/hour and .52 inches/hour, respectively, to a maximum and minimum of 6.00 inches/hour and 1.04 inches/hour, respectively. Thus, a wide variation in infiltration rates is represented. Only minor variance occurs at the beginning and end of the hydrograph. The peak is essentially unchanged. Although infiltration rate variability

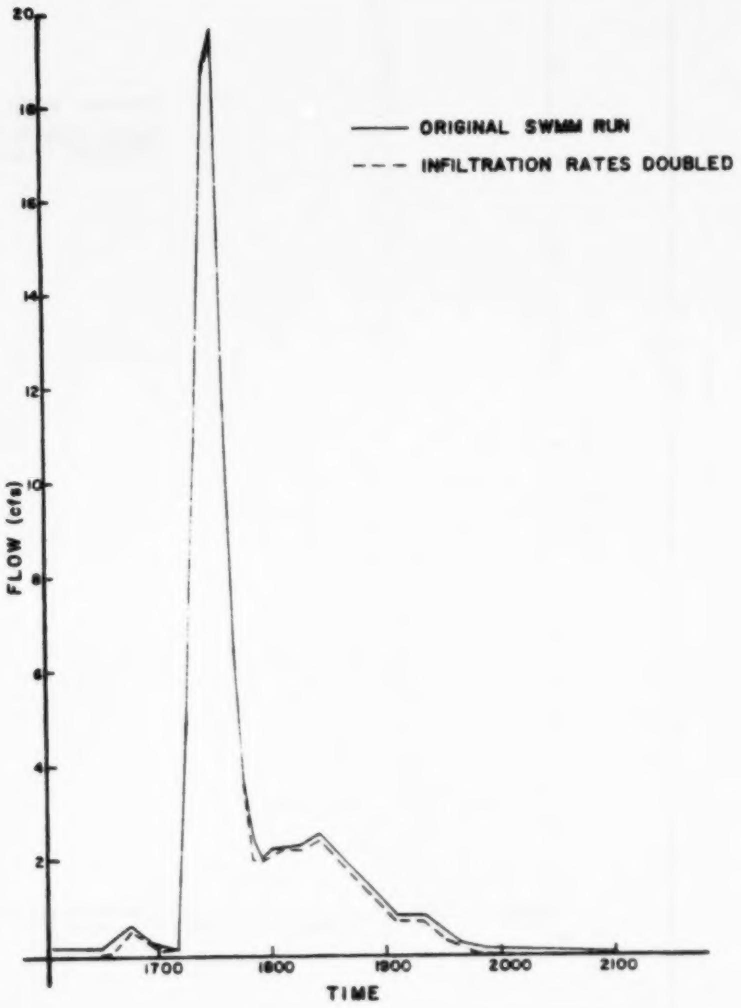


GRASSELLI CATCHMENT

RAINFALL DATE
JULY 15, 1976

FIGURE 1

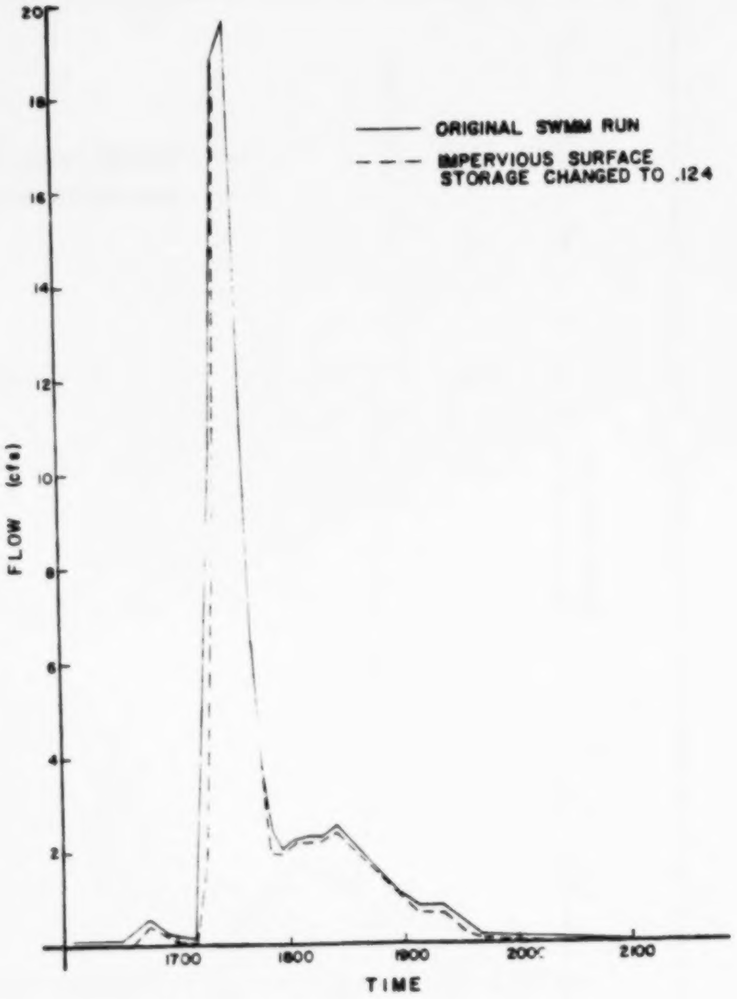
SENSITIVITY ANALYSIS
OF SWMM FEATURES



GRASSELLI CATCHMENT

RAINFALL DATE
JULY 15, 1976

FIGURE 2
SENSITIVITY ANALYSIS
OF SWMM FEATURES

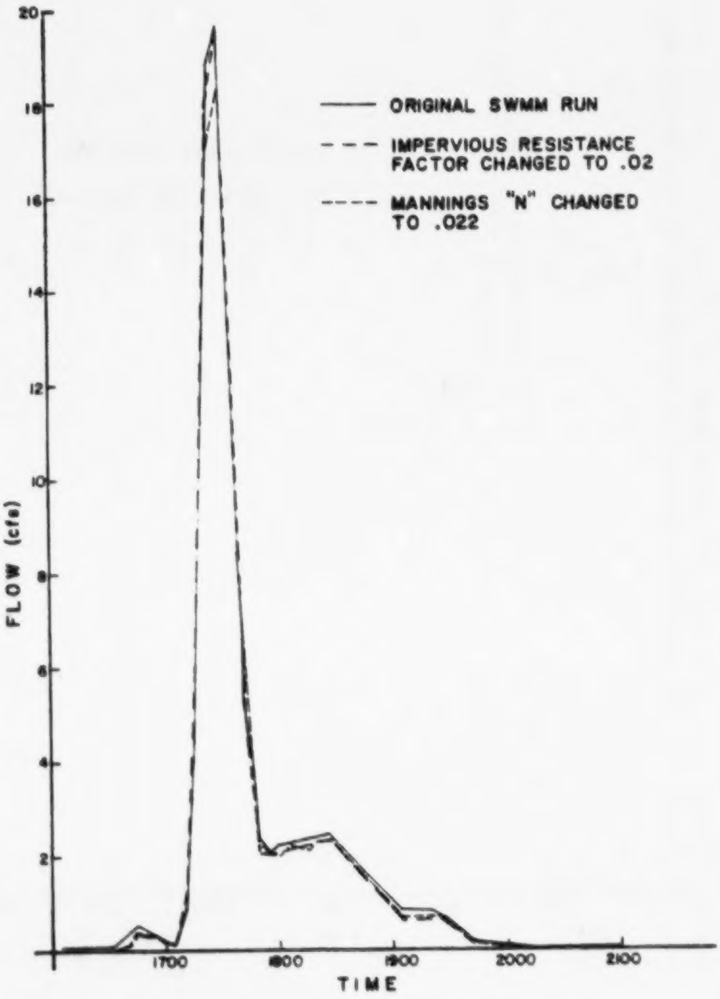


GRASSELLI CATCHMENT

RAINFALL DATE
JULY 15, 1976

FIGURE 3

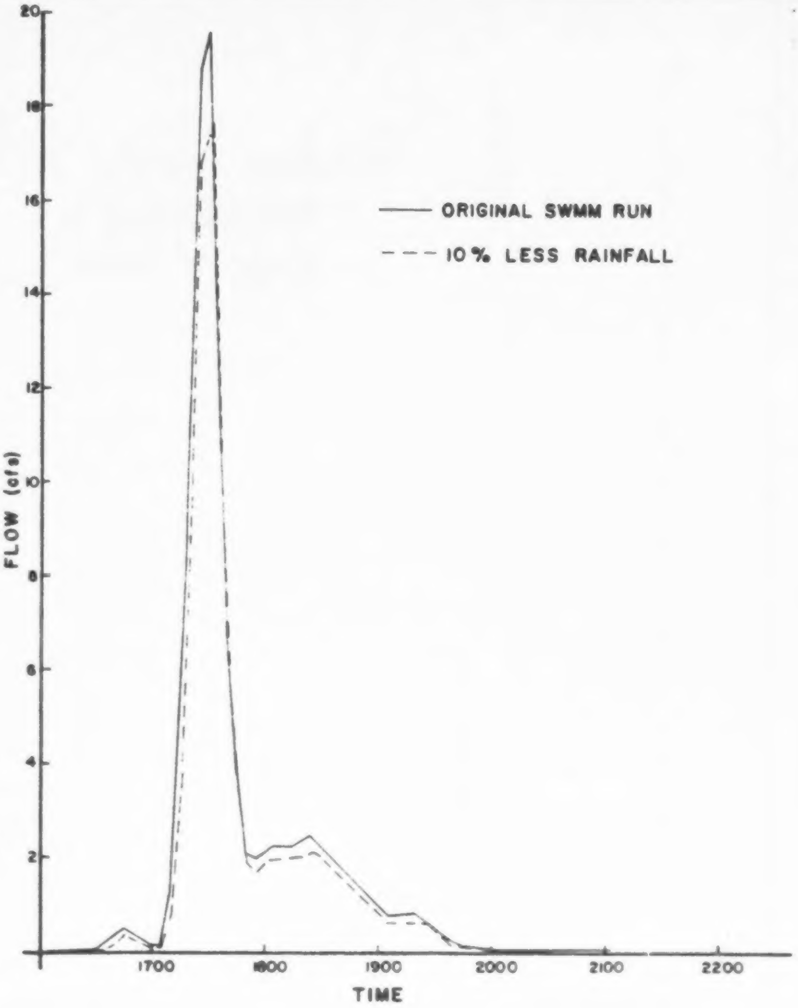
SENSITIVITY ANALYSIS
OF SWMM FEATURES



GRASSELLI CATCHMENT

RAINFALL DATE
JULY 15, 1976

FIGURE 4
SENSITIVITY ANALYSIS
OF SWMM FEATURES

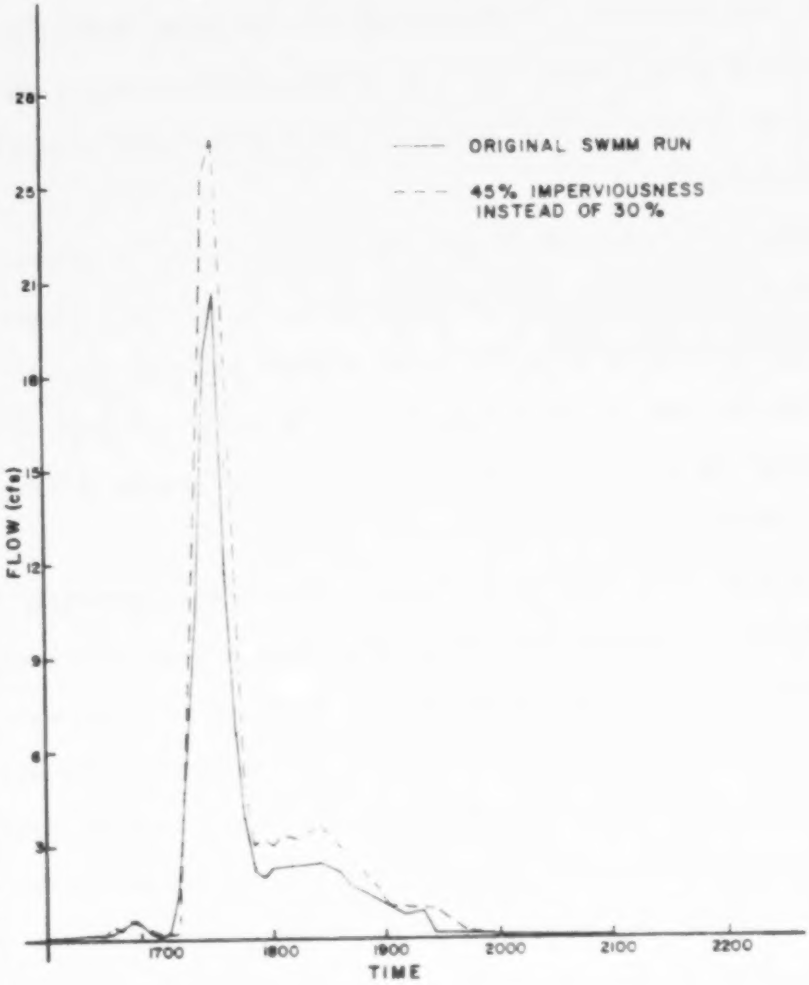


GRASSELLI CATCHMENT

RAINFALL DATE
JULY 15, 1976

FIGURE 5

SENSITIVITY ANALYSIS
OF SWMM FEATURES



GRASSELLI CATCHMENT

RAINFALL DATE
JULY 15, 1976

FIGURE 6
SENSITIVITY ANALYSIS
OF SWMM FEATURES

may have a greater effect in areas characterized by flatter slopes or a greater pervious percentage, it did not effect this study since the majority of land in Cincinnati is of an impervious characterization nearly equal to or greater than Grasselli and is featured by a highly dissected topography.

In Figure 3, a comparison is made between runs using an impervious area surface storage of 0.062 inches and doubling it to 0.124 inches. Here, absolutely no effect on peak runoff is shown and only minimal variations are apparent at lower flow rates. The variation apparently only occurs when rainfall accumulations are not great enough to completely fill all available impervious area surface storage.

It is apparent when looking at Figure 4 that if the impervious area resistance factor is changed from the program default value of .013 to .02, no significant change exists in the hydrograph definition, even though this change represents a significantly rougher overall surface. When Manning's "n" for the sewer elements were changed, however, from .015 to .022 the peak flow rate dropped significantly, being greater than five percent. This is due to a delay in the routing through the sewer elements downstream to the outlet and would be even more significant when analyzed on a larger watershed.

Data on Figure 5 shows that rainfall input significantly affects runoff quantities. Here, a ten percent reduction of rainfall intensities throughout the duration of the storm results in approximately ten percent less runoff volume and a reduction of the peak flow rate by ten percent.

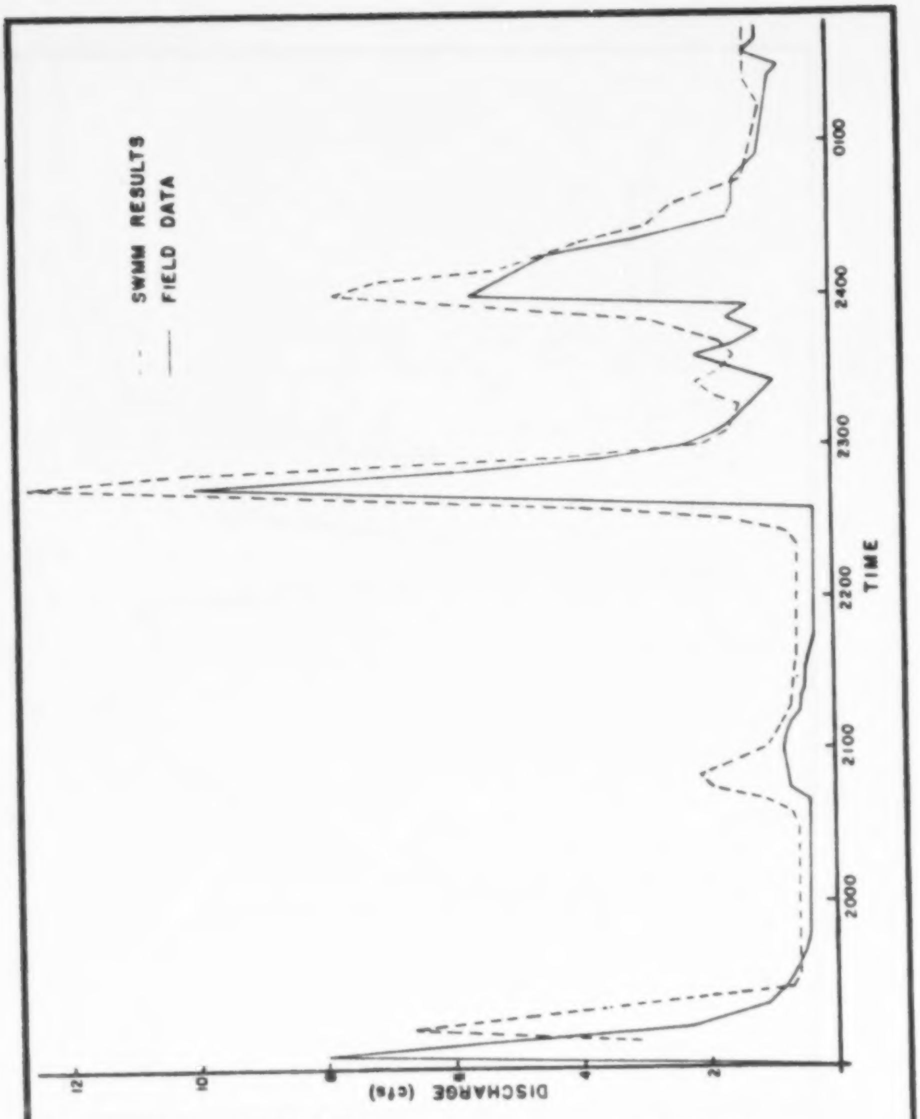
As was mentioned in the literature, the most significant parameter was surface imperviousness. Figure 6 shows hydrograph variability when the impervious percentage is increased by fifteen percent from an overall thirty percent to forty-five percent. Results show a significant change. Peak flow rates increase from 21 cfs to nearly 27 cfs and volumes also increase significantly. This verifies the previous findings that the percent imperviousness is a very significant parameter, if not the most significant in the calibration procedure.

Ignoring for a moment the characteristic width and weighted slope, all calibratable parameters in addition to those analyzed in the previously discussed figures were deemed insignificant. The pervious area resistance factor would be of less significant value than the impervious area resistance factor since it controls less flow. Pervious area surface storage would reasonably not be any more significant than the impervious area surface storage by the same manner of reasoning. If the maximum and minimum rates of infiltration are not significantly affecting runoff, then it would be safe to assume that the decay rate of infiltration would also be of minimal consequence. All other parameters are those which are based on actual measurements such as catchment acreage, etc. These were, of course, not included in the calibration procedure.

Based on the results of the sensitivity analysis made so far, it was concluded that as long as correct rainfall data were input, the only parameter affecting runoff calculations significantly enough to warrant change in the calibrating procedure was the impervious percentage. This premise is cognizant of the importance of Manning's roughness coefficient for the pipes through which routing occurs. The Manning roughness coefficient, however, should not normally be subject to wide variation. This intermediate conclusion, based up to now only on the Grasselli data, concurs with the results of the cited literature.

The percent impervious values were originally measured directly from aerial photographs. The area of all impervious surfaces (roofs, streets, sidewalks, driveways, etc.) were measured and divided by the total surface area of each subcatchment. Initial results for each of the rainfalls plotted, plus others that are not displayed in this report, predicted flow rates consistently higher than observed field measurements indicated. Basic shapes of the hydrographs were, however, very close to the shapes of the measured data. This indicated that the discretization procedure, routing flows through the pipe network, and input rainfall intensities were realistic. Subsequent runs were made lowering the impervious percentages. Finally, by lowering impervious values by five to ten percent, it was found that peaks and volumes correlated.

Comparisons of measured field data and SWMM simulations at Grasselli are shown in Figures 7, 8, and 9 for rainfall events dated June 18 and 19, 1976, June 24, 1976, and July 27, 1976. The first two plots were the result of rainfall events lasting over six hours, whereas the last plot represents the runoff results for a shorter duration rainfall. SWMM predictions are the result of using default values for all parameters except the impervious percentage. Routing was achieved through the Transport Block without any Runoff Block gutter/pipe elements. Correlation of SWMM results with observed data was excellent.

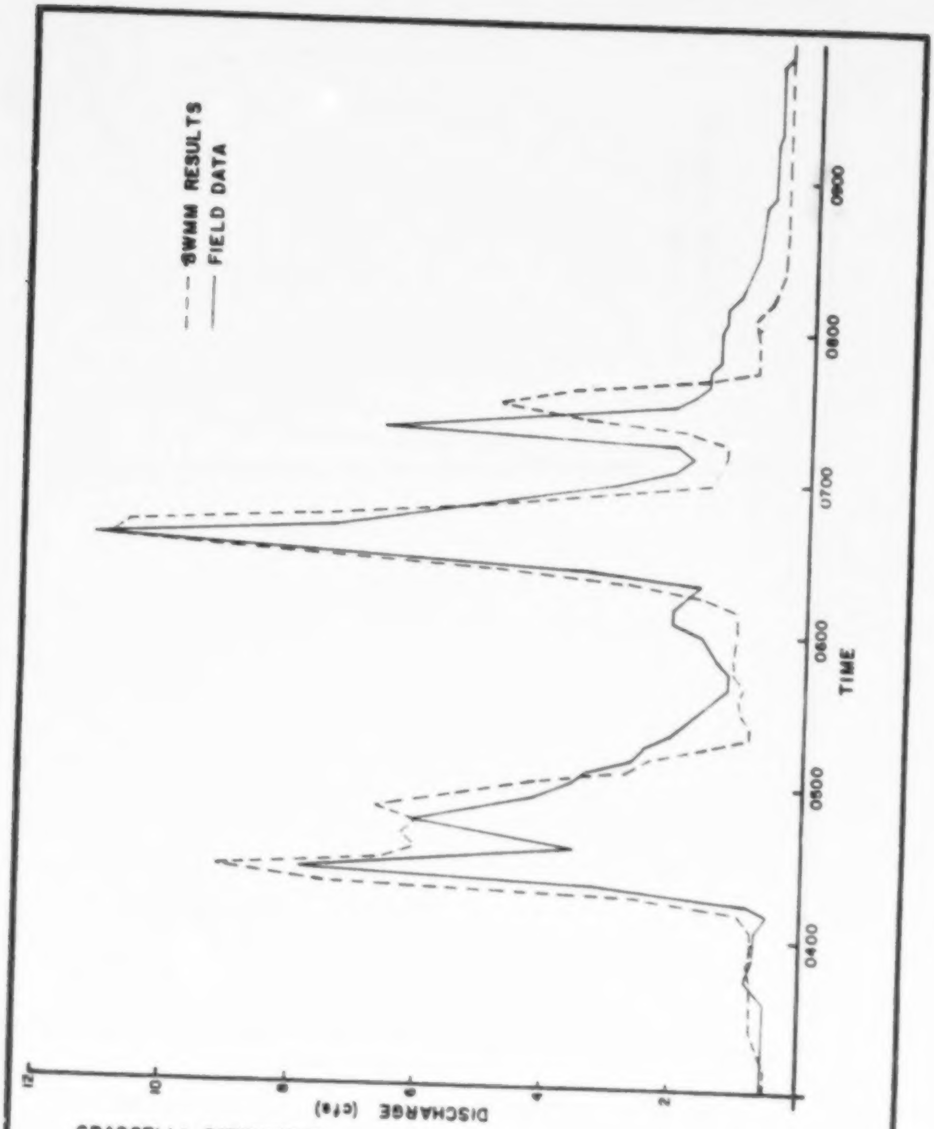


GRASSELLI CATCHMENT

RAINFALL DATE
JUNE 18 & 19 1976

FIGURE 7

COMPARISON OF SWMM RESULTS VS. FIELD DATA

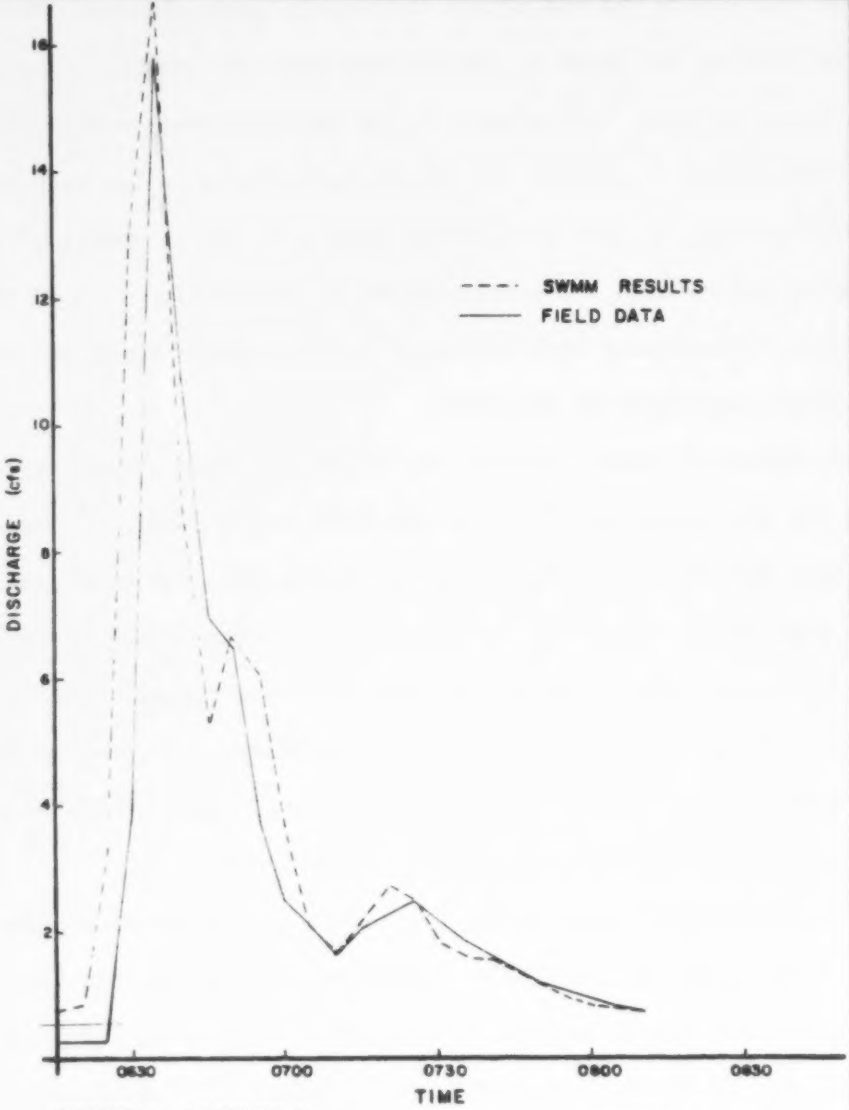


GRASSELLI CATCHMENT

RAINFALL DATE
JUNE 24, 1976

FIGURE 8

COMPARISON OF SWMM RESULTS VS. FIELD DATA



GRASSELLI CATCHMENT

RAINFALL DATE
JULY 27, 1976

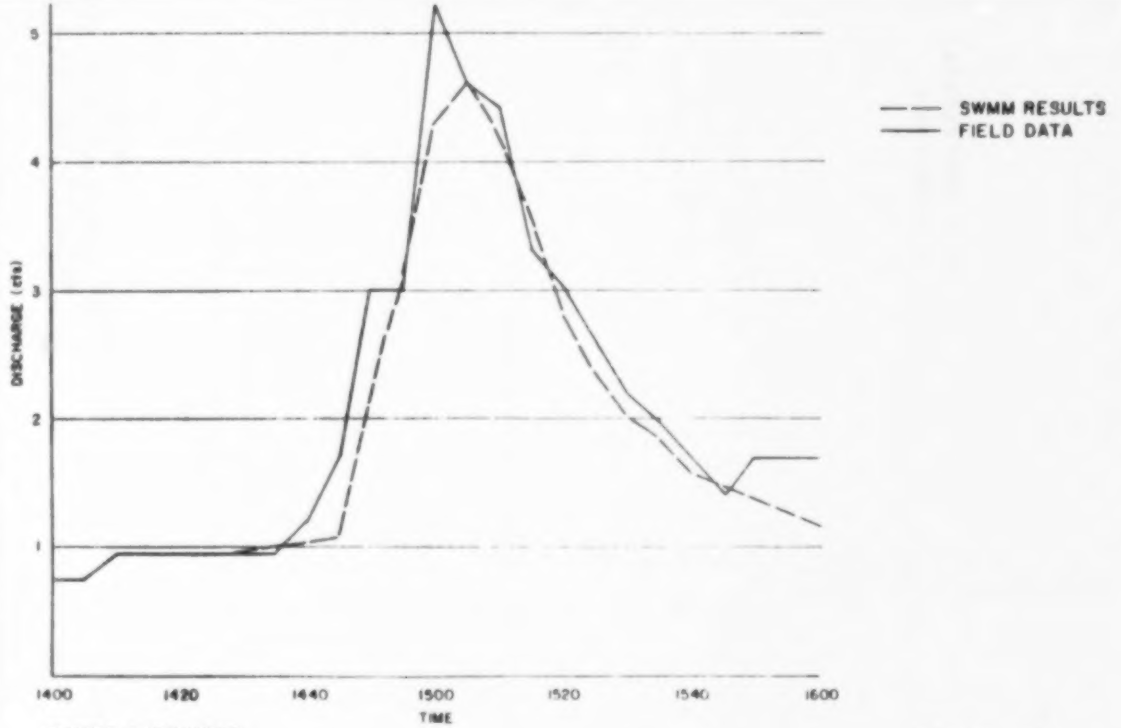
FIGURE 9

COMPARISON OF SWMM
RESULTS VS. FIELD DATA

Next, the Bloody Run and Sunset Catchments were analyzed. The purpose of testing the model on these catchments was twofold. A further analysis could be made with respect to the imperviousness calibration and the model could be checked for the correct procedure for calculating the characteristic widths and weighted slopes of subcatchments of a more complex nature than the subcatchments in Grasselli. In both these catchments, subcatchments were of a much more irregular shape and non-uniform slope than those of Grasselli.

Calculations of characteristic widths and weighted slopes strictly followed the procedures set forth in the SWMM User's Manual (4) and the Short Course Material (5). The predicted hydrographs showed a similar shape to that of the field data except that volumes and flow rates were somewhat greater. Impervious percentages were then reduced by five percent and the model predicted much more accurately. Figures 10 and 11 show a sample of the results using the rainfall of June 3, 1976 at Sunset and the rainfall of September 26, 1976 at Bloody Run.

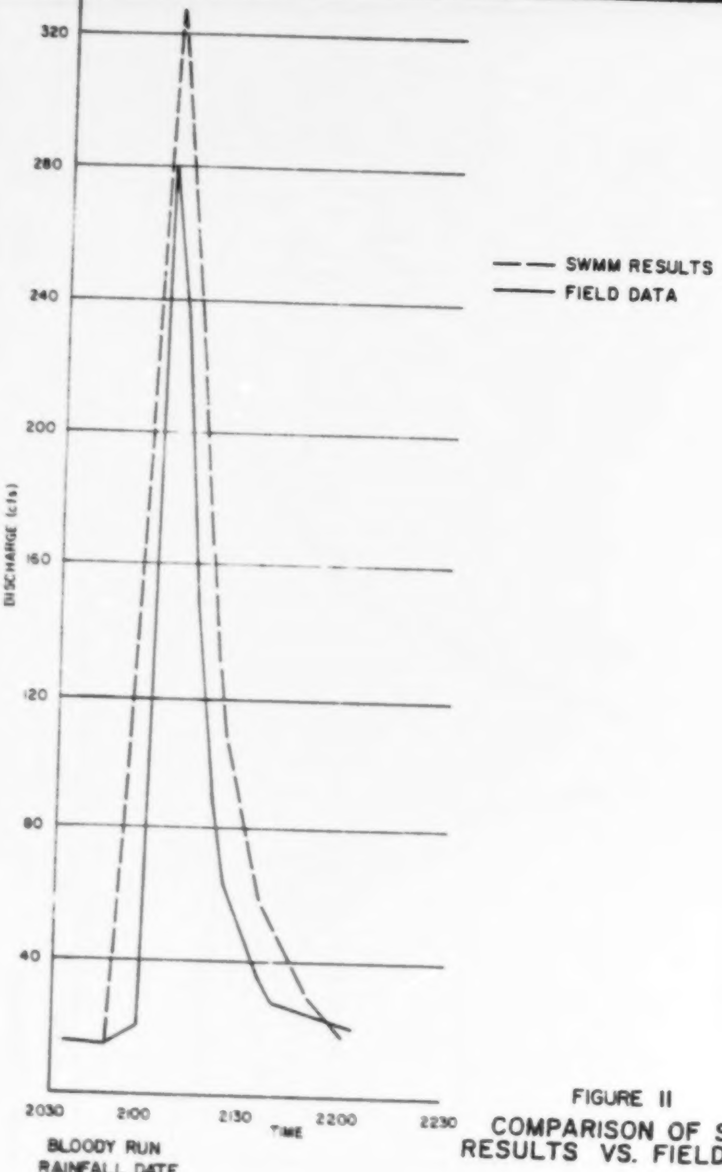
The model runs on these larger catchments led to two final conclusions. The characteristic width and weighted slope calculations, although variable, may be calculated adequately by precisely following the procedures set forth in the SWMM User's Manual (4) and the Short Course Material (5). Also, a percent impervious value of approximately five percent less than the measured value consistently yielded the best simulation results. This latter phenomenun can probably be best explained by the argument that, although impervious, certain areas may react as though they are pervious. For example, runoff from sidewalks and driveways may, at times, run onto pervious areas instead of into street gutters and



SUNSET CATCHMENT

RAINFALL DATE
JUNE 3, 1976

FIGURE 10
COMPARISON OF SWMM
RESULTS VS. FIELD DATA



BLOODY RUN
RAINFALL DATE
SEPT. 26, 1976

FIGURE II
COMPARISON OF SWMM RESULTS VS. FIELD DATA

drains; and garage roofs and occasional house roofs may, at times, drain out onto splash pads which discharge also onto pervious areas.

Based on this premise and actual measurements made from aerial photos, Figure 12 was devised. It is intended for use as a simplistic calculation procedure for estimating impervious percentages from dwelling unit densities. Three curves are displayed in the figure. The results of numerous measurements from aerial photos were fitted to the curve on the right. Here, all driveways, walks and garage roofs were included as impervious. The curve on the left side of the figure represents the remaining impervious percentage when all driveways, walks, and garage roofs were excluded. The median of the two curves represents the line which fits runoff simulations best and is equivalent to assuming that half of the driveways, walks and garage roofs act as impervious areas when calculating runoff, while the other half, draining onto pervious areas act as pervious areas when calculating runoff. The curve is intended to be used exclusively for residential areas. Direct measurements must still be made to ascertain impervious percentages from areas with other types of land use.

Finally, the model was verified based on comparisons of simulated hydrographs with field data for additional rainfall events at the three test catchments using the percent imperviousness plot displayed in Figure 12.

SIMULATION OF THE HYDRAULICS OF OVERFLOW STRUCTURES

Only the most simple types of overflows can be simulated with the SWMM via flow dividers. These are confined to two situations: (1) where

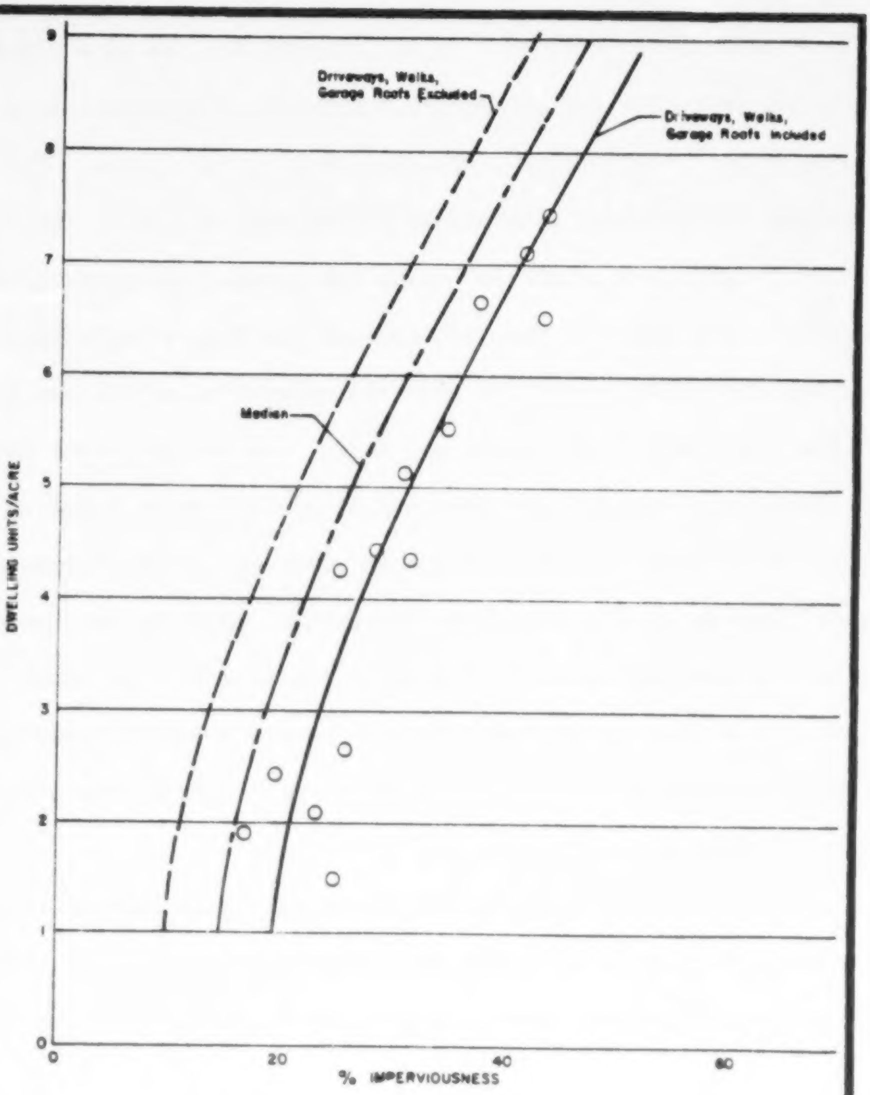


FIGURE 12

% IMPERVIOUSNESS vs. DWELLING UNITS/ACRE inflow is not diverted until it reaches a specified value and when the inflow exceeds that value the non-diverted flow remains constant and surplus flow is diverted, and (2) where a simple weir-type diversion structure can be modeled as long as flow rate and depth of flow into the structures can be linearly related.

Three general types of overflows were found to exist in the study area: Type "A", Type "C", and uncontrolled.

- The Type "A" overflow is a float and gate controlled regulator. As inflow increases due to runoff during a rainfall event, the float rises in an adjacent chamber connected to the trunk sewer via a "tell tale" pipe. The float, in turn, lowers the gate as it rises ultimately shutting of all flows into the interceptor at some predetermined level in the trunk sewer. At this point, all trunk flows overflow into the receiving stream.
- The Type "C" overflow is also a float and gate controlled regulator. In this case, the float is activated by the intercepted flow after it passes through the gate and before it is discharged to the branch pipe. As the float rises, the gate aperture decreases. However, this regulator is designed so that the gate never completely closes, thereby allowing a predetermined quantity through the gate regardless of the rate of flow coming into the regulator via the trunk sewer. All additional flow is overflowed into the receiving stream.
- Uncontrolled overflows do not utilize a gate and float mechanism. Flows are diverted into a branch pipe to the interceptor with or without the implementation of a weir in the trunk sewer. At some point, as flow increases overflow begins. The intercepted flow will continue to increase as long as inflow increases.

It was found that very few of the overflows encountered could be accurately modeled with the SWMM. The placement of outlet pipes which discharge to the interceptors and the frequently encountered situation where no weir exists presents many situations where even the uncontrolled overflows are too complicated for the SWMM. There were no instances found where float and gate controlled regulators could be modeled. Also, numerous circumstances were encountered where overflow devices operated in conjunction with each other.

Because the facilities planning effort revolves primarily around the generation of pollutant loads discharged into the receiving streams, the quantification of overflow volumes is vitally important. Consequently, approximation via the SWMM subroutines was not considered to be an adequate representation, and a separate regulator model was developed. The purpose of this model was to accept SWMM output hydrographs as inputs into each device and produce specific overflow and interception hydrographs.

The regulator model that was developed contains two distinct sections. The first section calculates and produces a table which displays the characteristics of the overflow device such as is presented in Table 1. For a series of depth/diameter ratios, influent trunk flows and corresponding intercepted flows and overflows are produced. This gives the user a quick picture of the operation of the device. The second section of the regulator model accepts the influent hydrograph (generated by the SWMM for this particular study) and produces intercepted and overflow hydrographs via a step-by-step reading and interpolation of the graph developed in the first section.

It is important to realistically describe the present conditions of each overflow device. Physical details from drawings can hypothetically predict the outflow for inflow hydrographs. However, quite often changes

TABLE 1

EXAMPLE OF REGULATOR HYDRAULICS TABLE

Depth/Diameter	Trunk Flow cfs	Intercepted Flow cfs	Overflow cfs
.01	0.01	0.01	0.00
.02	0.03	0.03	0.00
.03	0.04	0.04	0.00
.04	0.12	0.12	0.00
.05	0.20	0.20	0.00
.06	0.27	0.27	0.00
.07	0.34	0.34	0.00
.08	0.42	0.42	0.00
.09	0.49	0.49	0.00
.10	0.56	0.56	0.00
.15	1.44	1.44	0.00
.20	2.30	2.30	0.00
.25	3.67	3.59	0.08
.30	5.01	3.62	1.40
.40	8.64	3.67	4.96
.50	13.12	3.73	9.40
.60	13.19	3.78	14.41
.70	23.41	3.83	19.56
.80	28.43	3.89	24.54
.90	32.77	3.94	28.83
1.00	33.11	3.99	29.12

have been made over the years since the facilities have been built, or lack of maintenance may create situations quite different from that which would be hypothesized in the office. Consequently, on-site inspection of the devices is essential. This would include physical measurements as well as gagings of influent and effluent flow rates.

A METHOD OF ESTIMATING OVERFLOW LOADS

One of the primary goals of this study was the assessment of combined sewer overflow loads and a search for cost-effective means of the removal of these loads from receiving streams. These goals can best be achieved by examining the time-related response of pollutant loading rates at combined sewer overflows for various rainfall events. This is accomplished by developing relationships of pollutant concentrations varying with time. The multiplication of the pollutant concentrations by the rate of runoff, which is in the form of a SWMM-generated hydrograph, will yield the pollutograph, a plot displaying the loading rate of pollutants versus time, for any give rainfall.

The time and monetary constraints of the study warranted limiting the number of pollutant parameters to be analyzed. It was decided that five-day BOD and suspended solids would be the two parameters to be studied. These two parameters were chosen since they have been used most frequently and are, therefore, the most familiar. They are indicative of the two major constituents of water quality namely, solids load and oxygen demand.

The SWMM model has the capability of simulating the wash-off of pollutants which have collected on catchment surfaces since the previous rainfall event and adding these loads to the sanitary loads already flowing through the combined sewers. The output of the simulation may be in the form of pollutographs at any location that is desired throughout the system. However, this portion of the model has not been well documented and several reports(6) (7) (8) have discussed the difficulties involved in achieving satisfactory simulation. As discussed by J. Marsalek (6) : "The SWMM model runoff quality simulations were found to be less satisfactory. Though the insufficient data prevent drawing any firm conclusions, it appears that the quality subroutine is not readily applicable to all urban catchments. The SWMM quality simulations should be treated with great caution, particularly if used for a selection of urban runoff control alternatives, or policy enforcement. It may require another one or two years of data collection before the SWMM quality subroutine can be fully evaluated for the feasibility of application on Canadian urban catchments." The basic reason for the difficulty of simulating runoff quality by the SWMM model is due to an insufficient data base regarding the accumulation of surface pollutants which have accumulated since the previous rainfall. Little data has been gathered to date and approximations are difficult due to the veritable plethora of parameters affecting this phenomenon. Other models which simulate runoff quality are based on the same general input.

Much more data have been collected over recent years regarding concentrations of pollutants at the outlets of combined sewer systems. Therefore, it was decided to use this much broader data base to develop plots of pollutant concentrations varying with time and to forego computer simulation. This approach was developed using a coordinated effort between a sampling program conducted in three catchments in the MSD jurisdiction and a literature review.

The sampling program resulted in data being primarily collected at the three locations - Grasselli, Sunset and Bloody Run. As was previously mentioned, these three catchments vary considerably in size and land use, but are characteristic of the study area. Automatic sequential samplers were employed and set to sample approximately 10 minutes after dry weather flow rates were exceeded. Samples were taken in 10 minute intervals and flow rates were obtained simultaneously.

Concentrations versus time were plotted from the field data for numerous rainfall events at each of the three sampling stations. Averages of concentrations at each 10 minute interval were calculated and plotted. Then, a comparison of the average plots from the three gaged areas was made (see Figures 13 and 14). The abscissa (time coordinate) of each average plot was first established based on time elapsed since the start of the rainfall. A second series of plots was also developed in which the time coordinate was established based on coordinating the time at which the runoff peak occurred. The second series of plots yielded a more accurate coordination between the gages. This becomes the logical conclusion if one considers the variation in sizes of the three watersheds (42 to 2,380 acres) and the resulting variation in time elapsed from the start of the rainfall to the occurrence of the hydrograph peak. By aligning the abscissas of the three plots with respect to the hydrograph peak one is, in fact, adjusting the plots so that the various

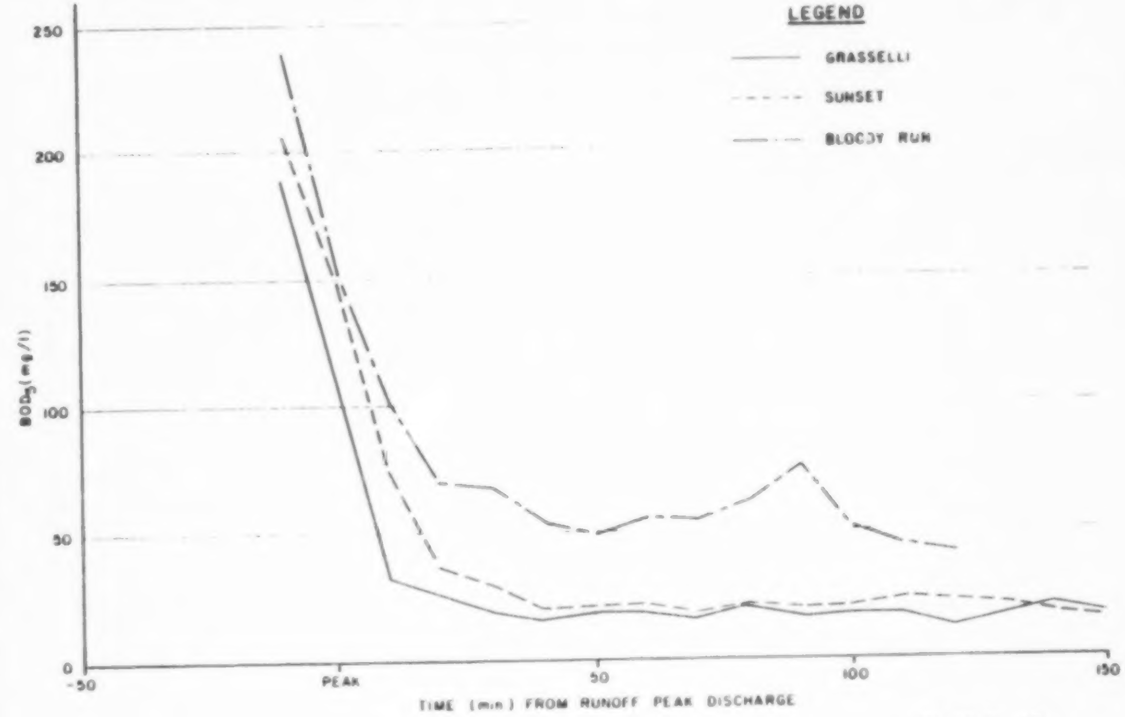


FIGURE 13

AVG. BOD_5 CONCENTRATIONS
(CINN. SAMPLING STATIONS)

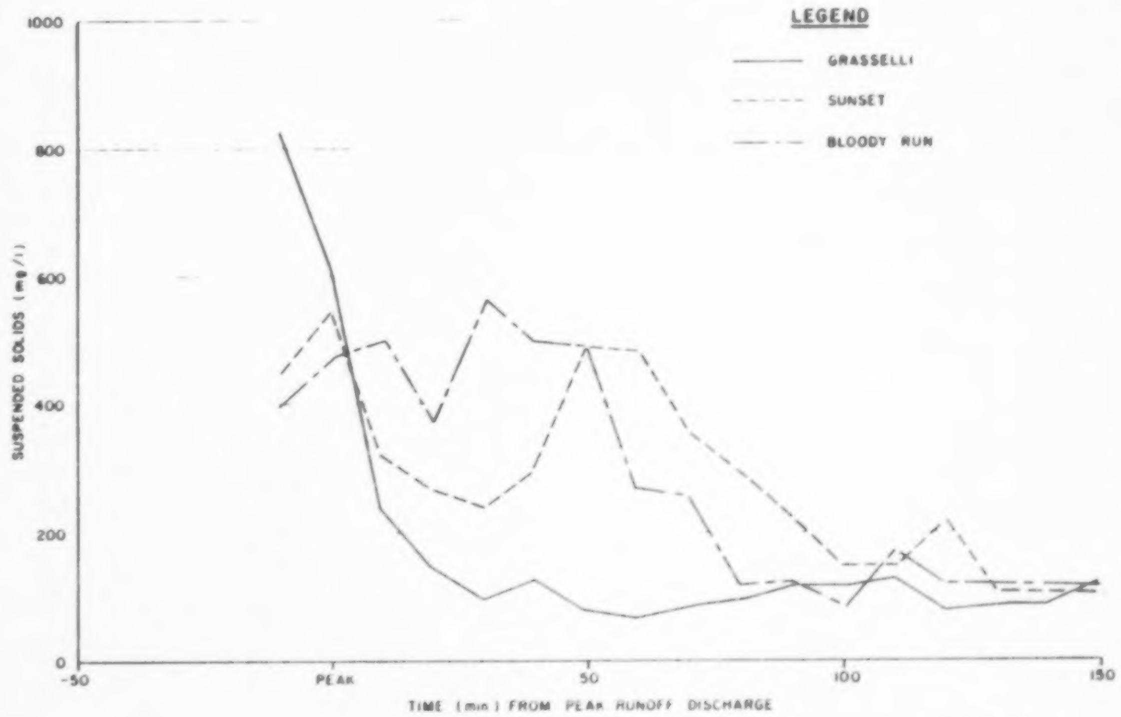


FIGURE 14

AVG. S.S. CONCENTRATIONS
(CINN. SAMPLING STATIONS)

phases of runoff concur best. This is considered to be the appropriate approach since the catchments to which the conclusions of the analysis are to be applied vary greatly in area, drainage configurations, and a resultant time of concentration.

Figures 13 and 14 illustrate that concentrations of both BOD_5 and suspended solids are higher during the early part of the runoff and decrease considerably as runoff continues. During the beginning of a rainfall, the solids which have accumulated on the surfaces of the catchment and have been deposited in the drainage elements, are washed off and scoured resulting in the relatively high concentrations. Eventually, most of these pollutants have been washed away and the concentrations of samples taken as the runoff continues are much lower. This confirms the already well-documented "first-flush" effect of stormwater runoff. The plots also illustrate a considerable variation in data due to both variations in catchment and to rainfall characteristics.

A literature review was conducted to supplement and verify the results of the sampling program. Literature data, in order to be useful, had to be able to depict the simultaneous concentrations of pollutants and flow rates at small time intervals throughout the duration of runoff for a rainfall event. Small time intervals (preferably 5 to 10 minutes) are necessary in order to establish an accurate portrayal of the variations in pollutant concentrations throughout the event. A thorough literature review revealed that published data generally lacks one or more of the basic ingredients needed for this analysis.

Through communications with Dr. Wayne Huber, University of Florida Department of Environmental Engineering Services, it was learned that a study was proceeding under contract with the U. S. EPA by the University of Florida, the purpose of which was to collect and study rainfall-runoff-quality data that is available throughout the United States and Canada. It was discovered that the data that was collected and tabulated conformed to the aforementioned criteria and was available via a computer tape. Upon receipt of the 9 track tape that was supplied, the data pertaining to combined sewer catchments were printed and tabulated. A description of the study areas for which data had been supplied revealed that seven catchments were drained by combined sewer systems. Data from all seven of these catchments were analyzed. The seven catchments include a 134 acre residential catchment in Lancaster, Pennsylvania, a 27 acre commercial catchment in Seattle, Washington, and 5 catchments in San Francisco ranging in size from 168 to 3,400 acres and varying in land use from primarily residential to primarily industrial.

As with the field data collected in Cincinnati, the data were plotted and average curves were developed for each catchment. The average curves were then plotted on one common graph and compared. Again, comparisons using the time scale (abscissa) relative to the occurrence of the hydrograph peak yielded closer comparisons than with plots using a time scale based on time since the start of the rainfall event. This correlation can be explained by the identical argument that was used for comparing the field data. The plots of these average curves are shown in Figures 15 and 16. Examination of these figures reveals that the results were concurrent with the results of the field data. The first flush is clearly depicted as is a noticeable scatter from one catchment to the next. Within each catchment, plots also varied considerably with varying characteristics of rainfall intensities.

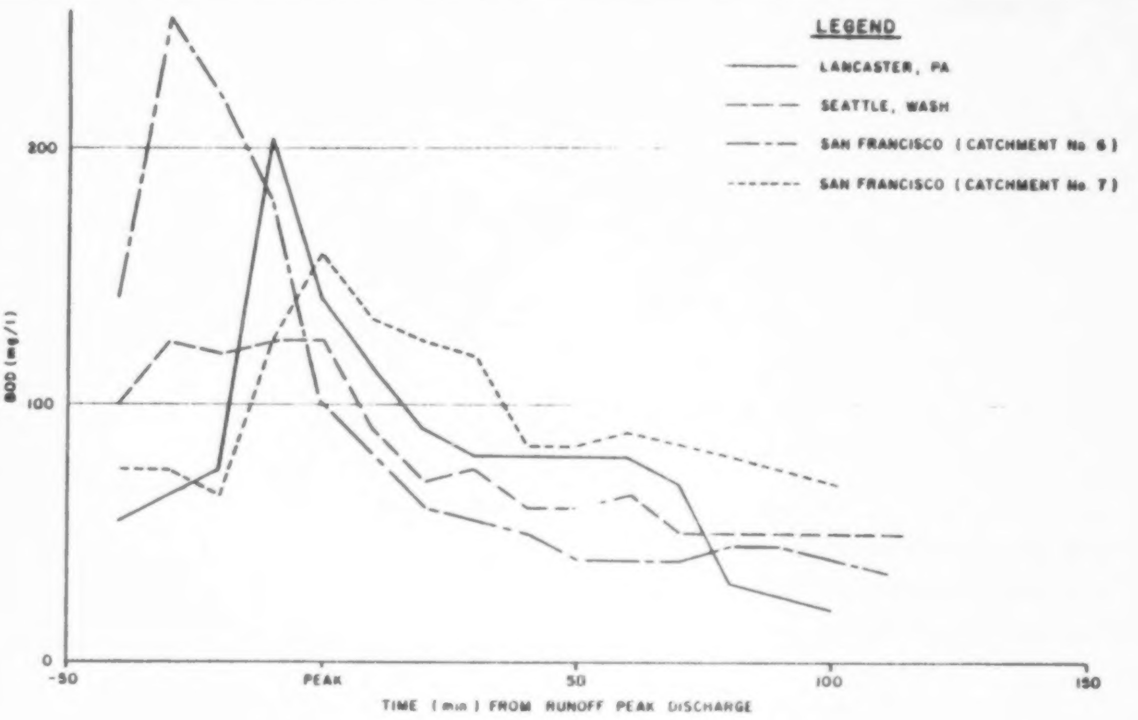


FIGURE 15
AVG. BOD_5 CONCENTRATIONS
(UNIV. OF FLORIDA)

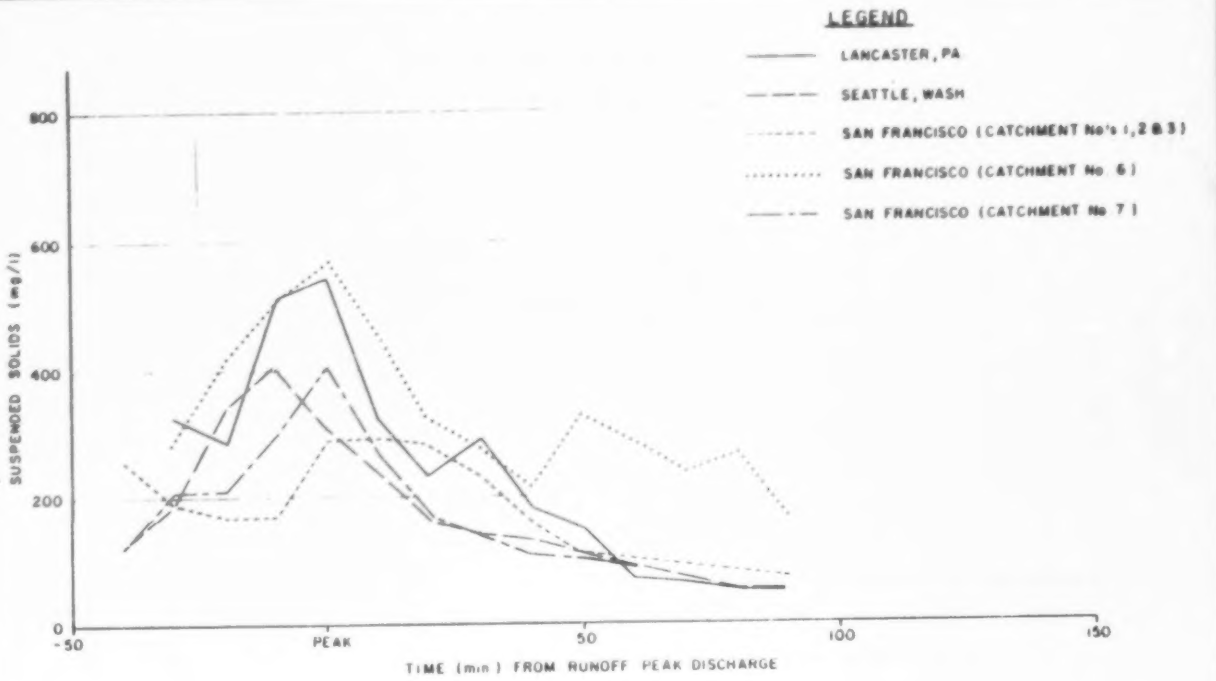


FIGURE 16
AVG. S. S. CONCENTRATION
(UNIV. OF FLORIDA)

The only distinct deviation between the University of Florida data and the Cincinnati field data was that concentrations were noticeably lower at the beginning of the event in the University of Florida data for both the BOD_5 and the suspended solids than those recorded during the occurrence of the runoff peak. Analysis of the rainfall events for the University of Florida data revealed that they were generally characterized by a longer duration and a lower intensity relative to the Cincinnati data, therefore, having a slower impact on the first-flush of surface pollutants and storm drain scour. Consequently, samples taken immediately after the beginning of the rainfall usually did not represent first-flush. Samples taken in Cincinnati, with rainfall represented by greater intensity, but shorter durations, almost always represented a first-flush effect with the first sample taken.

A recent publication by the U.S. EPA(9) discusses quality characteristics of first flushes and extended overflows obtained in Milwaukee, Wisconsin. Ranges are given for BOD_5 and suspended solids of between 170 to 182 mg/l and 330 to 848 mg/l, respectively for the first-flush and between 26 to 53 mg/l and 113 to 174 mg/l, respectively for extended overflow. These data were compared with the previous two sources for a final comparison.

Figures 17 and 18 compare the ranges presented in the U.S. EPA document with the ranges tabulated from the University of Florida data and the Cincinnati field data. Both figures illustrate the wide range of values that were obtained in all three sources. This is particularly evident when looking at the suspended solids range during the first flush. The ranges of values for extended overflows are markedly smaller

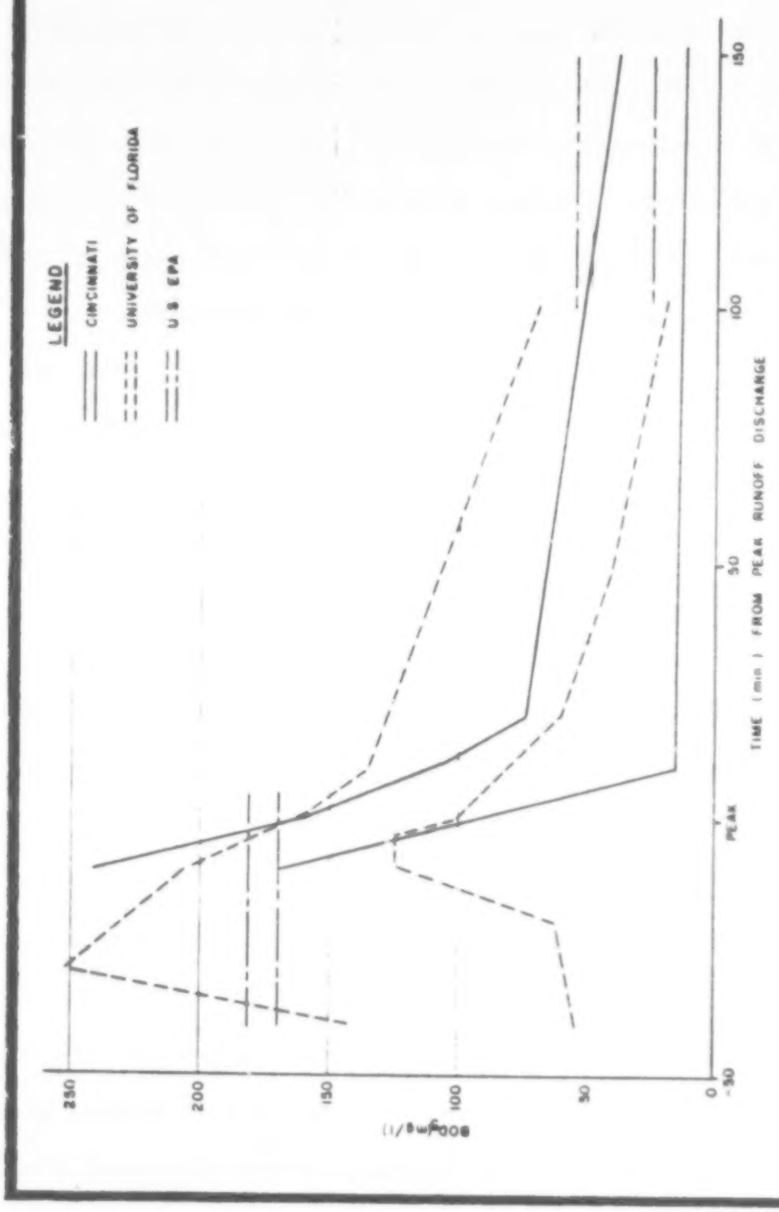


FIGURE 17
COMPARISON OF RANGES
OF BOD_5 DATA

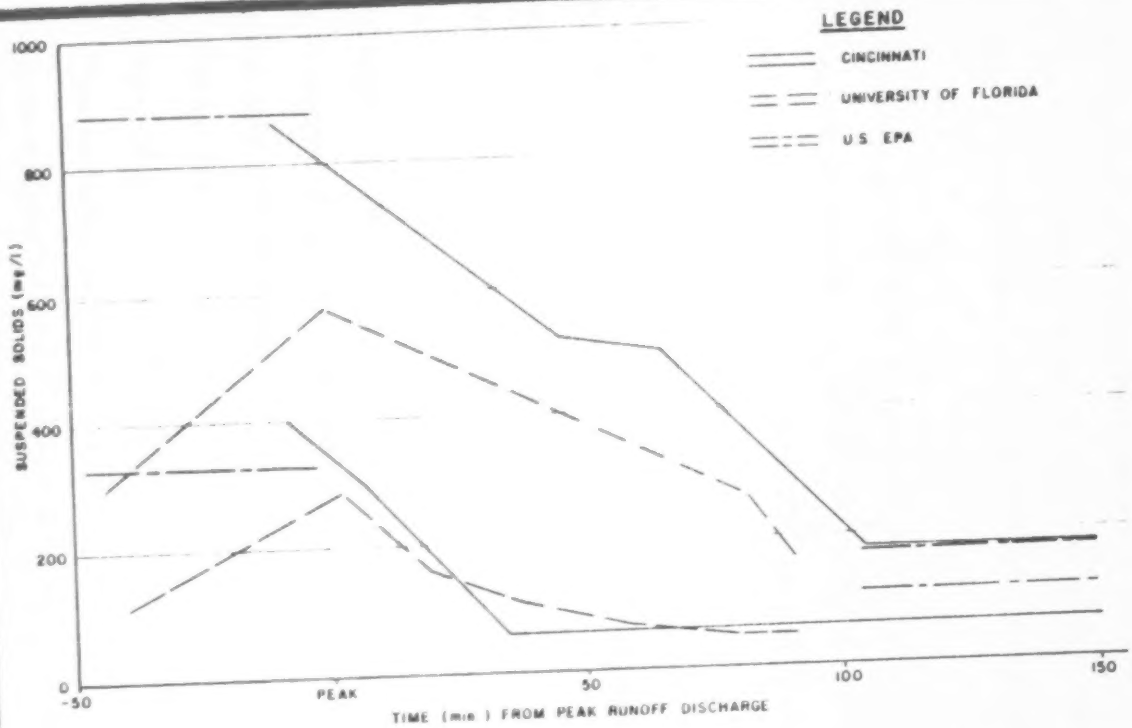


FIGURE 18

COMPARISON OF RANGES OF SUSPENDED SOLIDS DATA and correlate more closely between each of the sources. However, even though the ranges are considerable for each source, the same general pattern is apparent showing a maximum concentration at first-flush, before and up to the time of peak runoff, with a gradual dilution until concentrations remain constant throughout the remainder of the time of runoff.

With the analysis of the data presented thus far, it is evident that concentrations of BOD_5 and suspended solids are subject to a wide range of variations depending upon many different parameters. It would take a considerable effort to adequately analyze these data and tabulate the effects of each in order to develop an accurate pollutograph for every possible condition. Primary parameters that would have to be analyzed include catchment size, the shape of the drainage configuration, ground slopes, land use, catch basin details, fallout of dust particles carried via air currents related to the time elapsed since the last wash-off, rainfall intensities and rainfall duration. Other parameters, not mentioned here, may also have significant effects. This is a task that has not been accomplished by anyone to date. It is, therefore, important to recognize the approximations that have been made in this analysis when reviewing the conclusions to this water quality section. However, even though the conclusions are but a rough approximation, they were based on the most complete data-base available and would fulfill the objectives of this study to the limits of the present state-of-the-art.

Since a range of values will not satisfy the requirements for calculating the pollutographs, a specific concentration at each time interval had to be developed. The product of the pollutant concentration and the rate of runoff for each time step will yield the desired pollutograph. The best attempt at determining the final plot of concentration versus time was achieved by plotting the final averages from the three sources from which a representative curve was selected. This is shown in Figures 19 and 20.

The resultant representative curve for both BOD_5 and suspended solids was drawn from an approximate average of values among the three sources. In both cases, the first-flush concentrations were assumed to be constant from the beginning of rainfall until approximately 10 minutes prior to the runoff peak. These constant values correlate closely with the average value of the ranges set forth in the U.S. EPA publication(9) which concurred with the field data collected in Cincinnati. This was thought to be a better fit than averaging the University of Florida data since the nature of this study is to analyze runoff from critical rainfall events which would not be of the same nature as most the events for which data was collected by the University of Florida (i.e., a greater rainfall intensity at the commencement of the event would create higher initial pollutant concentrations).

The curve during the period between the first-flush and the extended overflow was developed as an approximate average of the University of Florida data and the Cincinnati field data. Minor oscillations were neglected. The extended overflow was a fit between the three sources and correlates closely with the average of the ranges presented by the U.S. EPA.

The abscissa of these figures may come under some criticism. Varying durations of runoff would affect the length of time the first-flush occurs as well as the period of time elapsed until concentrations remain constant during extended overflow. However, more detailed data are needed to make any further analysis. In most cases, the representative curve is intended to be applied to larger catchments, which would depict the most critical

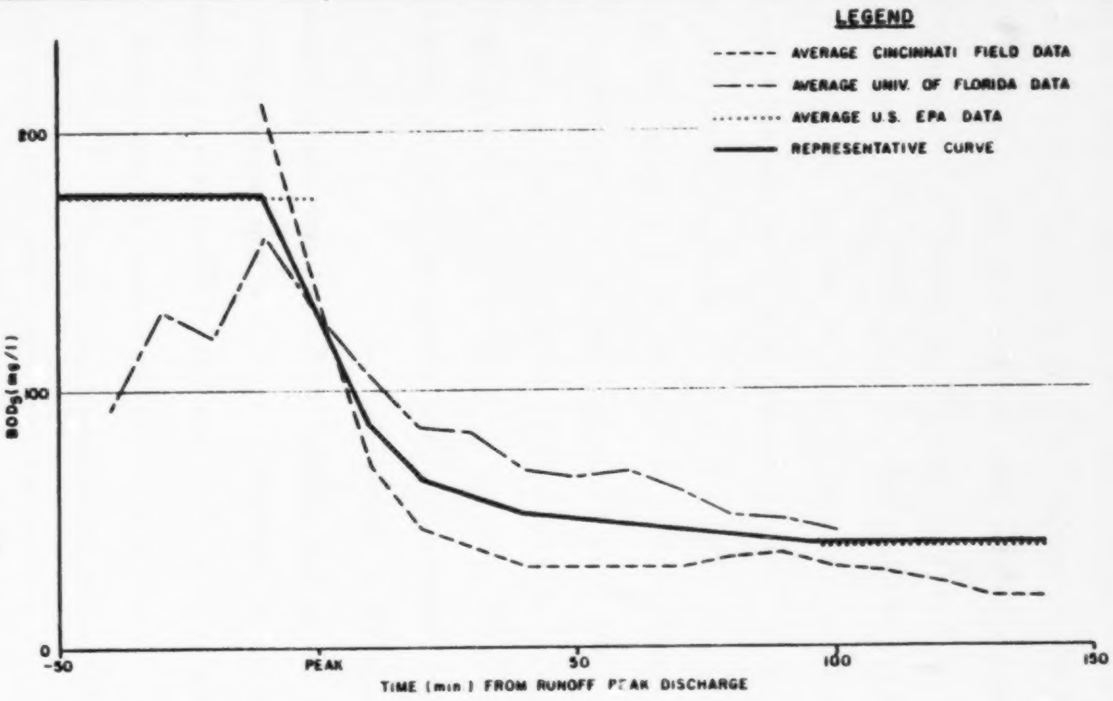


FIGURE 19
REPRESENTATIVE CURVE FOR BOD_5

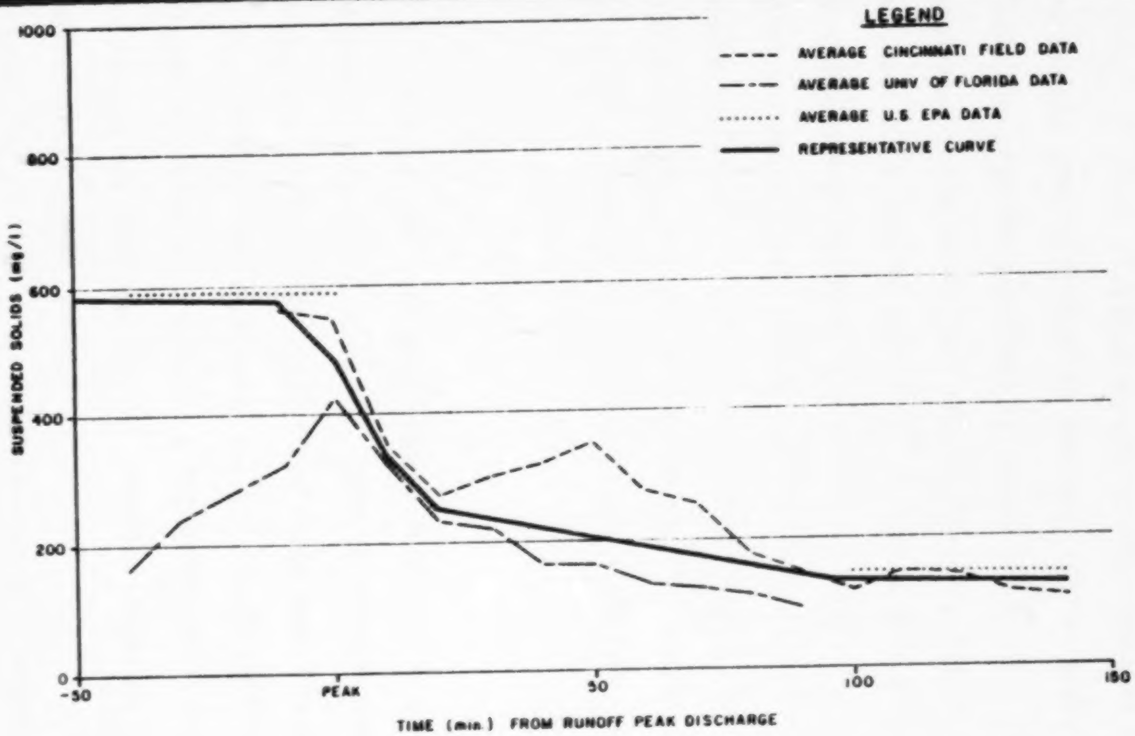


FIGURE 20
REPRESENTATIVE CURVE
FOR SUSPENDED SOLIDS

cases, and the time to peak runoff and extended overflow is closely represented by the time coordinate of the plot. It is certainly within the range of approximations carried forth in this analysis.

The final product of this analysis was a plot of BOD_5 and suspended solids versus time. It is the result of analyzing the most complete data base presently available, considering the time and monetary constraints of the study which place limitations on the amount of work to be done in the field. The final representative curves were combined with overflow hydrographs at key locations throughout the combined sewer system resulting in overflow pollutographs (loading rates versus time). The pollutographs were then used as a final tool for developing total discharges of pollutants from the system and analyzing alternatives available to reduce excessive discharge.

CONCLUSION

The SWMM served as the basic mathematical tool for the analysis of the Cincinnati combined sewer system. It was used as a simple hydrologic and hydraulic simulator upon which was built a more complicated hydraulic analysis of the combined sewer overflow structures and a qualitative assessment based on a literature search and a comprehensive in-field sampling program. The model calibrated quickly and was verified via repeated comparisons of output and field-observed hydrographs. Measured impervious percentages were altered so as to more accurately depict true runoff situations. The overall analysis procedure provided the basics for the quantification of pollutant loads generated in various alternatives leading to the development of the most cost-effective solution for the Facilities Plan.

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SWMM USERS GROUP MEETING

(Annapolis, Maryland, November 13-14, 19780

SWMM USAGE IN FACILITIES PLANNING

Abstract. The USEPA sponsored Stormwater Management Model (SWMM) developed in 1970 has been used extensively in Facilities Planning to simulate rainfall occurrences on urban drainage areas ranging from 5 acres to 1,600 acres. Resulting runoff hydrographs and pollutographs (showing suspended solids and BOD versus time) have been used to analyze the effects of rainfall on the quality and quantity of flows into collection systems, overflows, and flooding.

Case studies are cited which explain the various ways in which SWMM has been used in engineering studies, designs, and resulting or intended construction.

Extensive experience with the SWMM under a wide range of conditions has indicated that it is a valuable tool which can be used by urban communities of virtually every size to predict runoff occurrences for design rainfalls and actual rainfalls. Quantification of the runoff and its effects provide valuable data which can be used for engineering studies, design, and management decisions.

The authors are G.D. Cole, Senior Engineer, L.W. Varner, Engineer, and J.W. Shutt, Engineer, Floyd G. Browne and Associates, Limited, Consulting Engineers - Planners. Marion, Ohio.

Introduction

Floyd G. Browne and Associates, Limited, has used the SWMM extensively on a variety of projects ranging from villages which are primarily residential in nature to moderately sized cities with greater than 50,000 people and having substantial industry. SWMM was originally used during the facilities planning for development of data and eval uation of alternatives for transporting and treating the overflows from combined sewer systems. Modeled results have produced valuable information on the dynamic occurrences resulting in various drainage systems under selected storm conditions. Updated versions of the SWMM are constantly used as they become available with each version normally providing better modeled results. Improvements made in the SWMM hydraulic calculations (Release II, updated September 1976) have provided a tool for preliminary design of hydraulic structures. Table No. 1 lists the combined sewer system projects which this paper will discuss in detail.

TABLE NO. 1

LOCATION AND DESCRIPTION OF SYSTEMS ANALYZED BY SWMM

System	1970 Corporate Population*	Acres In Corporate Limits	Acres Modeled	Variation in Size of Individual Areas, Acres
Lima, Ohio	53,734	4,236	3,435	2-172
Bucyrus, Ohio	13,111	2,300	1,638	11-451
Hicksville, Ohio	3,650	900	708	5-50
Marion, Ohio	38,646	2,500	2,044	8-58

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*U.S. Department of Commerce, Bureau of the Census. 1970 Census of Population, Advance Report.

Lima, Ohio

Lima, Ohio is located in northwest Ohio approximately 50 miles southwest of Toledo and has a population of approximately 53,800; it is divided by the Ottawa River. Approximately 3,600 acres of the City are served by a combined sanitary and storm drainage system; an additional 3,500 acres on the periphery of the City have separated sanitary sewers and storm drains.

Problems of overflow from the combined system into the Ottawa River prompted a 1970 study to determine the necessary pollution abatement management necessary to alleviate this condition. In addition, during times of high river flow, water backed into the combined system via diversion structures and created a problem.

Because of the extremely high estimated cost of complete separation; the advance in technology as the result of increased research and development; and upgrading of stream quality criteria by the Federal and State authorities the City of Lima officials directed that the objective of the 1970 study for reducing pollution load in the Ottawa River be expanded. To be included in the increase was the development of a phased program with priority being given to the most cost-effective elements of the solution to achieve maximum pollution reduction based on National goals, including develop an alternative solution that would be less costly than complete separation.

Due to the numerous cross-connections and associated complexity of the City's combined system it was decided that a computer simulation modeling program would provide the most effective overall analysis of the systems' flow characteristics during any given storm. Modeling results of real storm occurrences were compared with field-monitored quantity and quality of flows to establish a correlation between the model and the actual system operation.

The watershed area for the 1970 study was 3,644 acres and was divided into subcatchments varying from 2 to 172 acres. Initially, the entire study area was modeled in a single program; however, it was decided to divide the drainage area into five main drainage areas so that the system could be modeled in more detail.

Figure 1 shows the relationship between monitored and computed results for flow occurring during the 1970 study at the downstream end of a major drainage area. This rainfall was moderate and major surcharging was not believed to have occurred; therefore, the Transport Block was able to adequately route the flows and interpretation to compensate for surcharging was not necessary.

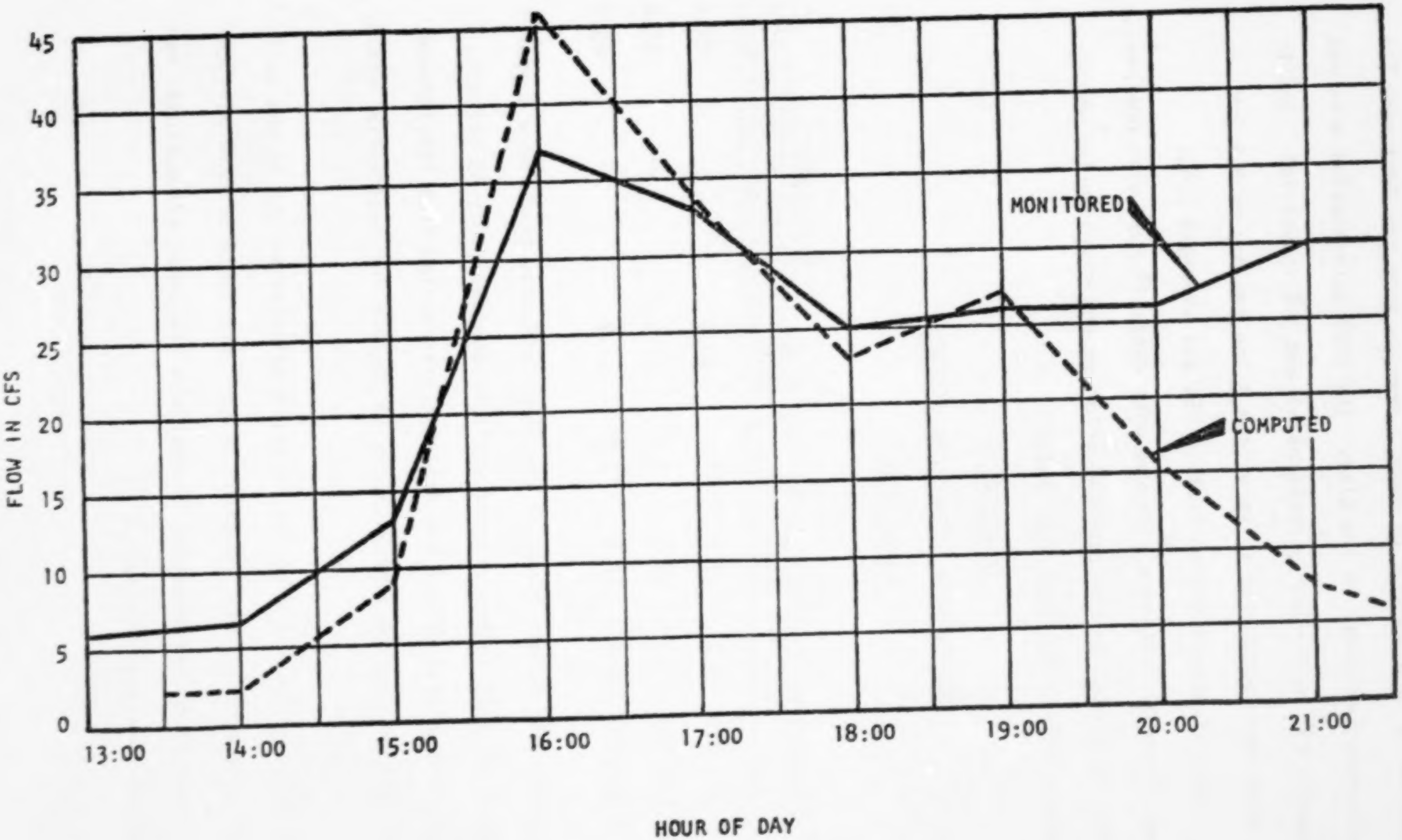

FIG. 1 Flow vs time at Collett - Spring for March 1, 1972 storm - Lima, Ohio

Table No. 2 is a summary of runoff results from the SWMM runs for some representative areas in the City. The 1327 acre modeled area was approximately 69% multi-family residential and 31% industrial. Infiltration rates were estimated to vary from 3 iph maximum to 0.5 iph minimum. Land slope varied from 0.2% to 35% and averaged 1.8%.
The street cleaning frequency used was once every 14 days with one pass, and 10 days were used as the interval prior to the storm during which accumulated rainfall was less than 1 inch.

TABLE NO. 2

RUNOFF DATA AS MODELED BY SWMM

LIMA, OHIO

Rainfall Intensity For 1 Hr. iph	Flow				Pollutants		
	Rainfall c.f.	Percent Runoff	Percent Infil.	Percent Storage	Suspended Solids Washoff (lb/acre) Residential	Commerical	Industrial
1.23	5,972,880	65.2	32.6	2.2	54.0	--	104.8
1.64	7,899,700	71.7	26.7	1.6	55.9	--	68.0
2.00	9,633,800	75.9	22.8	1.3	51.2	--	65.7

Generally, the Runoff Block of the SWMM indicated that for a 2 year-1 hour rainfall of 1.23 inches, runoff would be approximately 65 percent. Suspended solids washed off of the drainage area varied from 105 lb/acre for industrial land use to approximately 54 lb/acre for residential areas.

The process of street sweeping is given adjusted weight in the calculation of suspended solids. Because of the importance of this particular maintenance item, verification of the SWMM "internal assumptions" regarding street sweeping were made.

Typical of the design considerations that were investigated in this study was the selection of a design rainfall. It has long been taken for granted that the first flush of storm flow from combined sewers contains high concentrations of suspended solids and BOD. However, a search of the available published data provided virtually no specific design data on critical rainfalls. Accordingly, the runoff and washout of solids and BOD were calculated by the SWMM for the Lima system for numerous standard design rainfalls applicable to Lima. These were run through the computer program to determine the most critical storm durations and intensities for Lima.

The calculations confirmed that extra solids and BOD are "flushed" from the system early in the runoff period by all storms. However, it appears from the study that the flushing rate by a 30 minute rainfall is lower than that produced by longer or shorter ones. This means that more storage will be required to catch the washout from a 30 minute storm than if it were washed out by longer or shorter rainfalls. This phenomenon relates to the time of travel in the sewers; the shorter, more intense storms do not produce enough runoff to overload the collector sewers, so the solids are carried out rapidly. The runoff from the longer, less intense storms can also be carried by the sewers and the solids are washed out in a normal manner. The 30 minute storm, on the other hand, produces more runoff than the collector sewers can handle which causes surface flooding, thereby delaying the washout procedures. On the basis of this observation and reasoning, the 30 minute storm was accepted as the most critical for the Lima sewer system.

This does not mean that the 30 minute storm would be the most critical for all sewerage systems; the critical duration of rainfall would vary with (1) the capacity of the pipes with relationship to the area served, (2) the nature of the ground surface which affects the time of concentration and (3) the size of the community.

Most facilities are designed to operate at their maximum capacity at some predetermined frequency. Since the capacity of storm drainage facilities is influenced by rainfall, the frequency at which the design storm will occur is significant. It is not practical to design for the "ultimate" storm and the five year frequency is the most commonly used standard for the design of storm for the design of storm drainage facilities. The City of Lima uses the five year storm for the design of storm drainage facilities when their overload will not case basement flooding. The five-year frequency 30 minute rainfall for the Lima area as published in the U.S. Department of Agriculture publication, Number 204, is an intensity of 2.6 inches per hour or a total depth of 1.3 inches and this was used as the "design storm".

This study resulted in the recommendations of a two phase program for abatement of pollution in the Ottawa River caused by combined sewerage and storm water overflows. The Phase I project provides for controlled in-system storage with subsequent treatment of combined flows at the WWTP, while Phase II provides for detention structures at the overflow points.

These phases are explained in more detail as follows:

Phase I

1. Continue the current program to upgrade the quality of treatment by the wastewater treatment plant and increase its capacity in order to process controlled combined flow. The necessary capacities would be tertiary treatment for an average daily flow of 18.6 mgd with a peak rate of 33 mgd, and secondary treatment to 53 mgd.

2. Construct a new 54-inch intercepting sewer and lift station to route 53 mgd of flow to the WWTP.

3. Construct overflow and gate devices to program and control release of the combined flows to the intercepting sewer, the river, and the WWTP.

4. Provide a monitoring and automated control system to coordinate overflows, gate devices, the lift station and the WWTP.

Phase II

1. Install screens and pumps at the overflow points to lift the flow expected from the design storm at that point.

2. Construct storage structures on the river bank at the overflow points to hold the pumped flow. Equip the basins with mechanisms to remove the settled solids. Include overflow weirs

in the storage structures to permit the escape of excess flow to the river. Provide for returning stored liquid and settled solids to the intercepting sewer after the storm flows recede.

Figure 2 shows a comparison of the BOD and SS that are expected to be delivered to the WWTP through the existing system, a separation program, the Phase I facilities and the Phase II facilities. It was concluded that the City can reduce the escape of SS and BOD to the Ottawa River to one-half its present level with Phase I and essentially eliminate all escapes with Phase II.

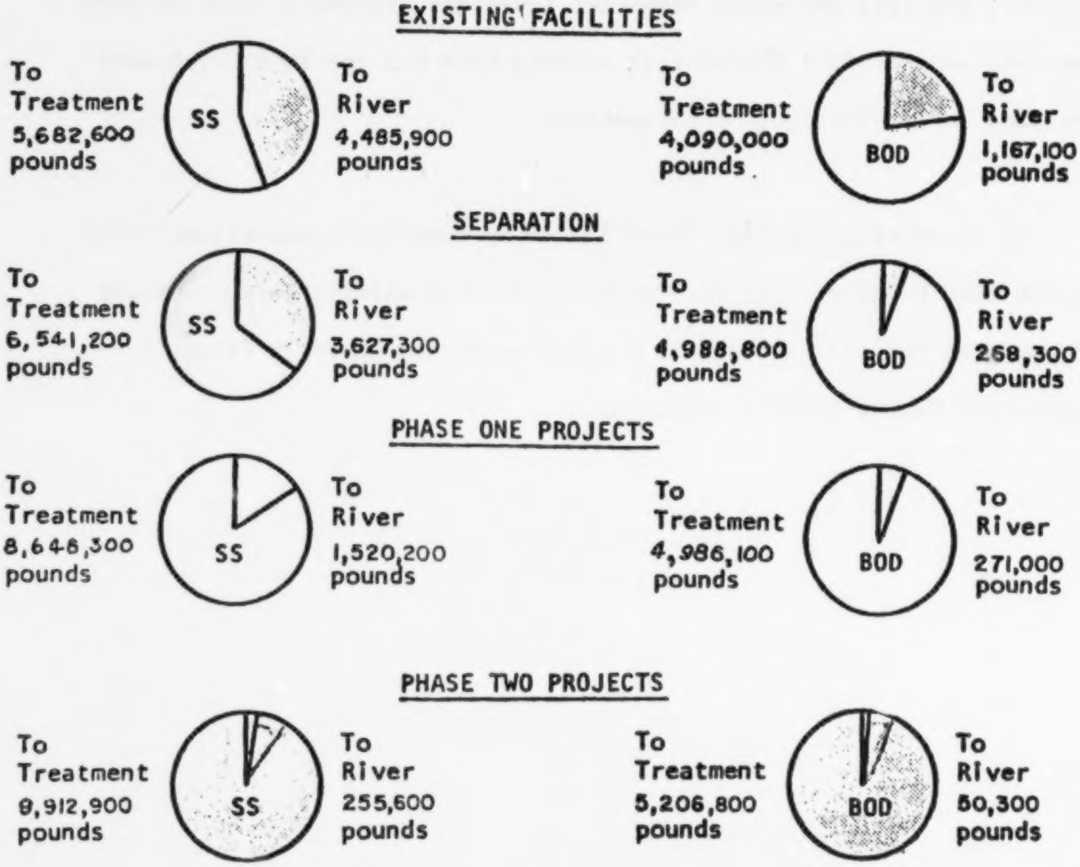
EXISTING FACILITIES
To Treatment 5,682,600 pounds
SS
To River 4,485,900 pounds
To Treatment 4,090,000 pounds
BOD
To River 1,167,100 pounds
SEPARATION
To Treatment 6,541,200 pounds
SS
To River 3,627,300 pounds
To Treatment 4,988,800 pounds
BOD
To River 268,300 pounds
PHASE ONE PROJECTS
To Treatment 8,648,300 pounds
SS
To River 1,520,200 pounds
To Treatment 4,986,100 pounds
BOD
To River 271,000 pounds
PHASE TWO PROJECTS
To Treatment 8,912,900 pounds
SS
To River 255,600 pounds
To Treatment 5,206,800 pounds
BOD
To River 50,300 pounds

FIGURE 2

The construction of Phase I was started in 1975. During the period of construction it became apparent that additional studies of the hydraulic effects in the system created by various rainfall intensities was needed to provide adequate information for automation of the system. For this analysis the Water Resources Engineers transport block of SWMM was used and rainfall intensities ranging from 0.3 iph to 4.4 iph were analyzed to set the control parameters.

It is anticipated that Phase I will be completely operational within the next 6 months. As for Phase II, it is presently being reviewed through the facilities planning process to determine if it is still cost-effective by today's standards.

Bucyrus, Ohio

The City of Bucyrus is located in Crawford County, approximately twenty miles west of Mansfield. The City had a population of 13,111 persons in 1970 and the estimated current population is approximately 13,300 persons. The corporate limits encompass approximately 2300 acres.

As part of an NPDES permit compliançe requirement, this City has completed a Facilities Plan, of which an Infiltration/Inflow Analysis is an integral part.

Alternative techniques for abating the pollution of the Sandusky River from combined sewer overflows were examined during the facilities planning stage. Alternatives considered were:

1. Provide separate sanitary and storm drainage systems.
2. Provide on-site treatment facilities to treat excessive combined flow.
3. Store excessive combined flow in aerated lagoons for subsequent treatment by the main wastewater treatment plant.
4. Construct on-site storage structures to detain the "first-flush of combined flow for subsequent treatment by the main wastewater treatment plant.

The final alternative proved to be the most cost-effective technique.

During the Facilities Planning stage an earlier version of the SWMM was used to predict the dynamic occurrences resulting from inflow. Data from that analysis were used to develop a preliminary design of an intercepting sewer and detention structure system for combined sewer overflow reduction. The purpose of that effort was to demonstrate the feasibility of the proposed system.

A detailed analysis of the percent capture of the various annual rainfall events and, therefore, percent capture of pollutants, assuming various detention structure capacities was conducted. Then a comparison of the cost per pound of pollutant removed by the intercepting sewer-detention structure-wastewater treatment plant system was made. The cost-benefit analysis showed that a detention capacity equivalent to the runoff resulting from a 0.3-inch rainfall is the most cost-effective.

It was decided that the intercepting sewer system should be sized to allow for future expansion of the detention structures if subsequent monitoring indicates that the receiving stream would benefit from such an enlargement. A 2 year-1 hour (1.23 inch) rainfall was selected as the design event for the intercepting sewer systems. This will allow transport of nearly all combined flow to the detention structure sites, and will insure economical expansion of the detention structures should that prove necessary.

The existing combined sewer system interceptor operates under surcharged conditions and the proposed system is designed to operate under a similar surcharged condition. However, the SWMM version used for preliminary analysis could not account for surcharging so its usefulness was somewhat limited. With the development of an improved model that could account for surcharging (Release II, updated September, 1976), it was decided to model the proposed interceptor-detention structure system again.

The preliminary design of the interceptors was based on the peak flows occurring at each of the 24 overflow locations. Also, these flows were assumed to be occurring under surcharge conditions resulting from the maximum water level in the two detention structures. This is obviously a very conservative design since it is not likely that all flow peaks will occur simultaneously or that they would occur against the maximum water level in the detention structures. However, there was no adequate method available to account for all contingencies prior to the development of SWMM. It is likely that a design based on the "worst case" a described above would have been recommended.

With the advent of SWMM, however, it is possible to design a less conservative system with the assurance that it will function properly and not introduce additional system flooding problems. SWMM was run initially with the pipes sized using the preliminary design assumptions. The output (flow through each pipe and water level at each manhole for each time step) was then analyzed for compliance with design objectives. The 0.3-inch rainfall was used to check the detention structure capacity and to minimize the possibility that no overflows would occur as a result of that event (0.6-inches/hour for 1/2 hour). The 1.23 inch-1 hour rainfall was used to check pipe capacities and that no increase in water levels in the existing collecting sewers as a result of the proposed system. An increased surcharge level in the existing intercepting sewer was allowed as long as the ground level was not reached and the water level in the existing collectors remained unaffected, since this would simply assure maintenance of maximum flows through the wastewater treatment plant at all times.

Table No. 3 shows a comparison of preliminary and final sewer sizes and lengths. Notice the shift to small diameter pipes that resulted from our SWMM analysis of this complex interceptor-detention structure system. The estimated construction cost for the interceptor system was resulted in [redacted] a savings of 25 percent as a result of the use of SWMM during the design phase of the project.

TABLE NO. 3

SEWER DIAMETERS AND LENGTHS

Diameter (inches)	Preliminary			Final		
	Length (ft)	Size Range	Percent of Total Length	Length (ft)	Size Range	Percent of Total Length
18	-	0-30	10.7	1,125	0-30	42.2
20	350			500		
30	850			2,930		
36	1,350	36-60	67.5	1,075	36-60	57.8
42	-			400		
48	3,805			1,810		
54	370			1,410		
60	2,060			1,560		
72	1,175	760	21.8	-	76	0.0
78	1,280			-		
Total	11,240*			10,810*		

*Minor changes in the total length are the result of adjustments in alignment after route surveys were completed.

Hicksville, Ohio

The Village of Hicksville is located in the southwestern part of Defiance County, in the northwest corner of Ohio. The population of the Village is approximately 3650 and it has a land area of about 900 acres.

This Village was modeled with the SWMM as part of the EPA grant program facilities planning process. The purpose was to analyse the effectiveness of the existing combined sewer system for transporting storm runoff and to evaluate overflow detention and treatment alternatives.

Hicksville's combined sewer system is arranged to carry most of the runoff south through town toward Mill Creek. In dry weather the wastewater flows into the major intercepting sewers and to the treatment plant. In wet weather, flow in excess of the treatment plant capacity is diverted to Mill Creek. A special problem at Hicksville is that a large area of undeveloped land to the west and to the north of the Village drains over land to Hicksville's combined sewer system and through it to Mill Creek. This problem was caused by the practice of covering over of Mill Creek as the Village developed. Now, runoff causes overloading of Hicksville's combined sewer system and ponding in several locations along the western and northern edges of the Village.

Hicksville is primarily residential in nature, however, there is some significant industrial development. Of 587 acres modeled, 45.0% is residential, 13.1% is commercial, 19.4% is industrial, and 22.5% is undeveloped land. Land slope varied from 0.1% to 3.0% and averaged 0.9%.

Rainfall intensities of 0.1, 0.3, 0.7 and 1.25 iph for 1 hour duration were modeled with SWMM to determine the resulting runoff and washoff of suspended solids.

The data in Table No. 4 indicate that the percentage of runoff increases with increased intensity. Also, for each land use the suspended solids washoff increases substantially with increases in rainfall intensity. Industrial and commercial areas have higher increases in washoff with increased rainfall intensity than do residential or undeveloped lands.

TABLE NO. 4

RUNOFF DATA AS MODELED BY SWMM

HICKSVILLE, OHIO

Rainfall Intensity For 1 Hr. iph	Flow				Pollutants Suspended Solids Washoff (lb/acre)			
	Rainfall c.f.	Percent Runoff	Percent Infil.	Percent Storage	Resid.	Comm.	Indus.	Undev.
0.1	212,270	23.3	55.9	20.8	0.36	1.76	2.32	0.39
0.3	636,832	37.1	55.9	7.0	3.7	16.1	23.9	3.6
0.7	1,485,870	41.1	55.9	3.0	18.1	79.7	116.8	16.7
1.25	2,653,350	47.7	50.6	1.7	37.6	190.0	260.7	51.4

Alternatives evaluated based on the SWMM data were sewer separation, primary treatment and chlorination of the overflows, and detention of the first flush produced by rainfall for later treatment at the wastewater treatment plant. The SWMM data for the four rainfalls modeled were plotted to establish flow and pollutant quantities versus rainfall amount. Analysis of rainfall records established the annual frequency of rainfall events of various depths in the Hicksville area. From these two data sources it was possible to estimate the average annual overflow and pollutant loading to the receiving stream under each alternative.

Detention of the first flush proved to be most cost-effective. Additional analysis was used to establish the most cost-effective detention volume. The ratio of the annual cost of the detention facilities which is composed of both amortization and O & M costs to the pounds of SS and BOD removed per year (cost-benefit ratio) was used for the evalua-tion. Table No. 5 shows these ratios for four different detention volumes. Selection of the cost-effective volume was based upon comparison to the wastewater treatment plant removal costs of $0.45/lb for SS and $0.40/lb for BOD. A favorable cost-benefit ratio exists for both SS and BOD up to 110,000 c.f. of detention volume; above that level the BOD cost-benefit ratio for detention is less than that for the treatment plant. Therefore, 110,000 c.f. of detention was selected.

TABLE NO. 5

COST - BENEFIT OF VARIOUS FIRST - FLUSH DETENTION VOLUME

Detention Volume (cf)	Annual Cost ($)	Pollutant Removal(lb.)		Cost of Removal($/lb)	
		SS	BOD	SS	BOD
85,000	65,000	510,000	200,000	0.13	0.33
110,000	80,000	540,000	210,000	0.15	0.38
155,000	100,000	510,000	215,000	0.18	0.47
340,000	205,000	720,000	230,000	0.28	0.89

Marion, Ohio

The City of Marion is centrally located in Marion County. U.S. 23 bypasses the city to the east in a north-south direction, 50 miles north of Columbus in the central part of Ohio. The City is moderately industrialized and has a population of about 38,700. A new 10.5 mgd wastewater treatment facility intercepting sewer and storm drainage projects have recently been constructed and a grant condition for these facilities was that the City study and correct the overflow and bypassing problems at various points in their combined sewer system. SWMM was used as a tool to assist in the evaluation during that study. Approximately 2,250 acres were studied. Land slopes varied from 0.1% to 1.7% and averaged

0.65%. Soil infiltration rates were estimated to vary from 3 iph maximum to approximately 0.5 iph minimum. Rainfall intensities of 0.2, 0.5, 0.9 and 1.23 iph for a 1 hour duration were used in the SWMM to determine the resulting runoff and washoff of suspended solids.

Completion of construction of the sewer projects resulted in additional first flush flows being transported to the improved wastewater treatment facility where they receive treatment. When these improvements are made, modeled overflows from a 2 year-1 hour design rainfall indicate that an improved overflow quality would result which is comparable to the quality expected from a storm drainage system if the City's system functioned as a truely separated system. A summary and comparison of the overflows expected from a separated system and from the existing system as being modified with the new projects is given in Table No. 6.

Coliform counts, as modeled by SWMM, in the overflows from the existing system modified with current construction projects are comparable to those counts expected in storm drainage. Increased coliform counts in the overflows are primarily a result of organic waste matter which is washed into the drainage system.

Although, Marion's system functions as a combined system, the overflow qualities will have been greatly improved with the new construction projects and additional treatment of these overflows is not expected to be cost-effective since the overflow to be treated has been modeled as having a quality essentially the same as stormwater.

TABLE NO. 6

EXPECTED FROM A 2 YEAR-1 HOUR RAINFALL
OVERFLOW QUANTITIES AND QUALITIES
CITY OF MARION, OHIO

		POLLUTANT DISCHARGE FROM STORM DRAIN IF A SEPARATE STORM DRAIN EXISTED, LBS.		OVERFLOW FROM EXISTING SYSTEM MODIFIED WITH CURRENT CONSTRUCTION PROJECTS		
				POLLUTANT, LBS.		FLOW, GALLONS
STORM SYSTEM DRAINAGE AREA	ACRES	SS	BOD	SS	BOD	
COLUMBIA DITCH	1,168	26,900	1,600	26,600	2,000	17,500,000
SILVER ST. & NILES ST.	876	35,400	2,100	37,200	2,800	16,500,000
FOREST LAWN BLVD. & MT. VERNON AVE.	201	4,900	350	4,900	410	3,100,000
TOTAL	2,245	67,200	4,050	68,700	5,210	37,100,000

Conclusions

The Storm Water Management Model can be used as a predictive model for studying various aspects of urban runoff. Both quantity and quality can be predicted on a dynamic basis and when interpreted properly can be valuable parameters by which the system can be evaluated.

The accuracy of the model is obviously only as good as the data which is fed into it as input. Various studies have been done which evaluate the input parameters according to their effect on the output. Using these as guidelines, it is possible to place the proper emphasis on the input so that the most precise information is obtained for the most effectual parameter.

Runoff quantities and qualities for various land uses can vary widely with the type of development. For example, newer industrial areas with large green spaces will not contribute as much per acre as an older industrial area with little or no green space. These parameters are important to consider for input to the SWMM and each modeled system should be individually defined with the best parameters possible.

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A PRE- AND POST-PROCESSING PROGRAM PACKAGE FOR THE STORM-WATER MANAGEMENT MODEL

by

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A PRE- AND POST-PROCESSING PROGRAM PACKAGE FOR THE STORM-WATER MANAGEMENT MODEL

by W. James*

__Abstract__: A new package called FASTSWMM has been developed and used by the author in Sweden and Canada. The basic program in the package is the standard SWMM program, distributed by the U.S. E.P.A. and is maintained by updating the basic SWMM modules in accordance with the latest modifications and corrections issued by the original authors. This basic program has in no way been modified by the present author.

The processes included in the package are:

a) solicit and accept user-directed initial SWMM input in conversational free-format mode, usually from a remote terminal
b) solicit and accept design-oriented input data (if any) from input devices or units as directed by the user for the evolving design
c) check the validity of the input and return values of certain dependent parameters to the user terminal
d) convert from metric units to imperial or U.S. units required by SWMM, if necessary, and reconvert to metric data if required
e) submit the blocks of SWMM in the order required
f) return selected output from the SWMM output file to the user terminal
g) re-submit the SWMM blocks as directed and return selected output, and repeat as required
h) save and list input/output files as directed by the user

Steps (e)-(h) are written in simple systems control statements and should be reasonably easily adapted to different systems. Steps (a)-(d) are handled by FASTSWMM programs. The package has been carefully designed to minimize user errors and reduce total design turn-around times. The paper describes the urban drainage design problem and FASTSWMM design criteria. The FASTSWMM package evidently considerably reduces the complexity of SWMM job submission; users are able to focus on the simple hydrologic, hydraulic and water quality processes modelled. No knowledge of FORTRAN formats, or systems control language is required of users. As such it is useful in an instructional/training environment and the paper describes such courses given by the author.

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INTRODUCTION

Urban drainage problems arise from both the quantity and the quality of runoff. So far as flood flows are concerned, urban runoff models are essentially similar to general hydrological models such as the Stanford Watershed Model (see Figure 1).

In practice there is little difference between flood control management and urban drainage management - we have to control excess runoff with minimum damage and cost.

The textbook approach to runoff control has been described by Linsley in several publications and this introduction is adapted from his work. Typically the approach requires determination of a flood frequency curve for the existing drainage condition and a flood frequency curve for conditions after completion of the proposed runoff control systems. These curves can be converted into damage frequency curves by assuming a relation between peak flow and flood damage. The area under the curves is the average annual damage in the existing condition and average annual damage after improvements have been made. The difference between these two damage figures represents the benefits of the flood control or drainage project and can be compared with the costs of the project for an economic evaluation. In many urban situations the damage is little more than nuisance and a decision is made rather arbitrarily to limit the probability of this nuisance to some acceptable level [Linsley and Crawford, undated report].

In dealing with pollution from urban storm runoff, the magnitude and frequency of the pollutant loads should also be known. A determination should be made of an acceptable level of pollution frequency given some information on the magnitude of the polluting load. In other words, the approach should be similar to that for flow quantities. Of course, rare storm events are not necessarily determining; long dry periods may be more significant.

Consequently, it is necessary in urban hydrology to define the probability of peak flows, and, where storage is being considered, the volume characteristics of the streamflow as well. Unfortunately, it is common to define flow frequency by selecting a rain hyetograph and converting this to a flow hydrograph with a discrete event simulation model. But the most intense rainfall in a year does not necessarily produce the maximum peak or the maximum runoff volume [Linsley and Crawford]. A short duration high-intensity rainfall may produce a very large peak flow but with a low runoff volume such that it will be severely reduced by available storage. On the other hand, modest rainfalls extending over many hours, or even days, may produce a large volume of runoff which could fill a storage reservoir. The exact effect will of course depend on the amount of storage and its outlet capacities.

The General Nature of Urban Drainage Design

It is important to discuss the reality of urban drainage design: it is an improvisation, a provisional lash-up. It is not possible to design a drainage scheme that works properly, or one that achieves a truly satisfactory performance. There is no storage facility that has no unfortunate or undesirable effects. The requirements for good drainage design are generally irreconcilable, and all designs are in a sense failures. Compromise implies a degree of failure, for example, durability vis-a-vis low first-cost.

The influence of economy in drainage is everywhere apparent; the characteristics of a "functional" design are usually derived from the requirements of economy and not of use.

The principle ways of reducing costs are:

a) to design and to construct the project as quickly as possible,
b) to use simple construction techniques,
c) to use as few operations as possible,
d) to use as little skill as possible in the work force.
e) to use as little material as possible,
f) to use the cheapest materials available,
g) to use the most easily worked materials,
h) to use the cheapest energy available.

The general design procedure in urban flood control may be thought of as a series of nested loops as shown in Figure 2 [Ontario MOE, 1977].

The urban drainage designer has limits set upon his choice:

1. The drainage facility being designed must correctly embody the essential principle or the fundamental use of the facility - a detention pond, for example, will accumulate fluctuating quantities of water. We cannot bluff our way around it.
2. The drainage components must be geometrically related as required in extent and position to each other and to the overall system. They will not disappear at some future data.
3. The components must be strong enough to resist the required hydrostatic and erosive forces, and big enough to convey the flow.
4. Access must be provided, for construction and maintenance.
5. Cost must be acceptable, at least to the majority.
6. Appearance must acceptably accord with the environment.
7. Side effects must be acceptable to those affected.

Hence, drainage design always involves making trial assumptions based on experience; it is a matter of trial and error and relies heavily on memory. A totally inexperienced person cannot design a good drainage scheme, even when using computer models.

Of course computer models are clever devices, but users sometimes seem to claim that the model will show us the correct and cheapest answer. However, there is no optimal design. Even if we were able to deliberately pre-determine mathematically the solution that would actually turn out to be the most economical in long-term practice, the public would probably still want something cheaper or more striking.

CURRENT URBAN DRAINAGE DESIGN PRACTICE

Fortunately Canadian design practice has been reviewed and described recently in several publications [MacLaren, J.F., 1975; Proctor and Redfern et al, 1976; Marsalek, J., 1976; Environment Canada, 1976; Ontario MOE, 1977]. This chapter is an adaptation and amalgamation of these publications.

The design of storage is now generally agreed to be possible only through the synthesis of hydrographs (storage is not necessarily concentrated in a reservoir but may be distributed over different elements of the watershed, such as roads, parking lots, roofs and elements of the sewer network).

It has recently become common to consider two extreme drainage systems in one urban area. One is a minor system consisting of closed and open conduits and the other is a major system, which is the route followed by flood or runoff waters when the minor system is inadequate. Traditional urban drainage studies considered only the minor system. Nowadays both extremes are analyzed in a drainage project. Typical minor systems are usually intended to have sufficient capacity to collect and transport runoff from a storm that might be expected to occur only once in a 2 to 10 year period. Return periods from 25 to 100 years might be considered for the major system [J.F. MacLaren Ltd., 1975]. Detention ponds which are part of the minor system are designed with consideration of possible effects of major floods. The same methodology for the hydrologic simulation should be applied to both systems.

SWMM flow simulation has proven to be accurate in a large number of applications for areas of greatly differing size, land use and sewer system configuration, and over a range of meteorological conditions. Consequently SWMM has been recommended for the simulation of flows, where required, in drainage planning and design studies, although particular advantages offered by other models in certain specific situations should not be overlooked. For example, STORM may be useful for the screening and comparison of alternatives and for the isolation of critical events. [Proctor and Redfern, 1976].

In the planning stage of the design, various land-use alternatives, drainage systems and the resultant pollutional impacts on the receiving waters are evaluated [Marsalek, 1976]. Typically, at this stage only limited information describing the watershed is available and, consequently, a detailed runoff simulation would not be feasible or appropriate. At the same time, it is important to establish the probability of occurrence of runoff events of specific magnitudes. The SWMM model could be applied as a spatially lumped model without a significant sacrifice in the accuracy of simulation [Environment Canada, 1976]. Reduction of the number of elements in the RUNOFF and TRANSPORT blocks to a minimum results in a considerable saving in data reduction time and in computer time requirements.

Hydrographs and pollutographs are generated by SWMM for the different runoff (or overflow) control alternatives. At the completion of the planning stage, the user has a good indication of the nature of the runoff or overflow problem in the study area and also has a feeling for the effectiveness of various runoff/overflow control measures for a number of critical rainfall/runoff events. The information obtained during this phase is at a level where the relative effects and magnitudes are more important than the absolute values required for subsequent design purposes.

In the next stage of the design, the design of drainage system and pollution control facilities is carried out, as well as a detailed study of probable impacts in the receiving waters. Consequently, it is desirable to produce accurate hydrographs and pollutographs for selected events using a calibrated, detailed simulation model. At this level, SWMM would be used at a high level of discretization. Surcharging becomes very important when analyzing methods to reduce flooding in an existing sewer system of insufficient capacity, or when evaluating the response of a drainage system to a storm of a frequency lower than the design frequency.

At present there are few suitable continuous surcharging simulation models available for the kind of urban drainage design described in all of the previous pages. One possibility is to use a tried-and-tested model such as SWMM in a flexible manner. The development of FASTSWMM is an attempt to provide rapid and easy use of the SWMM package in this kind of design environment. It is an attempt to introduce new users to complicated programs such as SWMM without having to learn software or programming related material. Hopefully the FASTSWMM philosophy will be extended to include other proven hydrologic and hydraulic packages.

DESCRIPTION OF THE FASTWMM PROGRAM

FASTSWMM makes it possible to run parts of SWMM from a terminal in a pseudo-conversational mode. FASTSWMM was originally developed by the writer in Sweden and known as SWESWMM. The program is presently available on the CDC Cyber computers at Multiple Access, Toronto and through the TELENET system to almost all users in North America.

FASTSWMM comprises three parts: PRESWMM, SWMM or ZAPSWMM, and POSTSWMM (see Figure 3).

The basic processes included in the package are:

a) solicit and accept user-directed basic SWMM input in conversational free-format mode, usually from a remote terminal

b) solicit and accept design-oriented input data (if any) from input devices or units as directed during execution by the user

c) check the validity of the input and return values of certain default and dependent parameters to the user terminal

d) convert from metric units to the British Units required by SWMM, if necessary

e) submit the blocks of SWMM in the order required

f) return selected output from the SWMM output file to the user terminal, and reconvert to metric data if required

g) calculate simple indicators of the correlation between the calculated and observed (if any) hydrograph, and printer-plot both hydrographs.

h) re-submit the SWMM blocks as directed and return selected output, and repeat as required

i) save and list input/output files as directed by the user.

Steps (a)-(d) are handled by the pre-processor, steps (f)-(g) by the post-processor. The whole procedure has been carefully designed to minimize user errors and reduce total design turn-around times. The FASTSWMM package evidently considerably reduces the difficulties of SWMM job submission; users focus on the simple hydrologic, hydraulic and water quality processes modelled. No knowledge of FORTRAN formats, variable names or systems control language is required of users.

Figure 3 also indicates how the package is set up for a multi-processor environment: the pre- and post-processor reside on a demand processing CPU while SWMM resides on a big number-crunching batch processing CPU. The general philosophy from the point of view of the user is shown in Figure 4.

SWMM is the main part of the program. FASTSWMM was originally tested on a stripped down version of the Canadian version of SWMM (CANSWMM), called ZAPSWMM containing only the flow hydrograph parts of the RUNOFF block. ZAPSWMM does not contain routines for snowmelt, water quality or plotting of the results and requires only 48 K_{10} words of core. SWMM or ZAPSWMM is automatically submitted to a central processor unit as a remote batch job by the package.

PRESWMM works in an interactive mode and prepares the input for SWMM. PRESWMM makes it possible for the user to enter his input in free format, and in metric units.

POSTSWMM also works interactively. It returns selected results to the user and reconverts the data to metric units. It also computes the linear correlation coefficient between the computed hdyrograph and the observed hydrograph (if any), and printer plots the two hydrographs.

After the normal log-in procedure, FASTSWMM is activated from the terminal by the command USE, FASTSWMM.

From then on, control passes to the program. The user only has to enter data as required by the program. However, the user may optionally take over control by entering a command instead of data.

The following are the valid commands:

BACKSTEP	(start this step again)
DEADSTART	
END	
CHANGE	(change an alterable variable)
HELP	

In a similar way to the HYMO program (Williams and Hann, 1973) and the HEC programs, commands are expressed in at least the first 3 columns of the data line, and columns 4 through 80 are used for numeric data and keywords. The commands are started in the first space and may be curtailed after the third letter; only 3 letters are necessary to identify the command.

The data can be written in any format, but at least one blank space or comma must be left between data items. A decimal is required for numbers containing fractions, but not for whole numbers. Keywords can be written with the data to describe individual data items. When entering data one should first leave 3 spaces blank (these spaces are for the commands). Spaces or commas are used to separate data. When all data corresponding to one prompt are entered, the line ends with a "carriage return".

For example four of the first few prompts are as follows:

i) IS YOUR SWMM DATA METRIC OR FPS ...

? FPS (or METRIC)

ii) 4. ENTER I/O ASSIGNMENTS ...

? 0 (if the runoff block only is to be used)

iii) 5. ENTER SCRATCH TAPE ASSIGNMENTS ...

? 1 (necessary for the runoff block)

iv) 6. ENTER BLOCK TO BE CALLED ...

? RUNOFF

When prompts for entering data are printed at the terminal they are followed by a reference to "CARD" n. The necessary "CARD" descriptions are the same as in the SWMM documentation.

Where computed flows are to be compared with observed flows the following should be noted:

a) Observed flows must be stored on a file on the user's account.

b) Computed flows and observed flows must be in the same units.

c) It is necessary for the response to the prompt (concerning observed flows)

```
AT WHAT LOCATION ...

?
```

to specify a location for which flows are being computed by the program.

To make a run with revised data, the user proceeds as follows:

1) Get the file NNNINPX (NNN = user's initials, X = case number) from permanent storage (NNINPX is produced by a normal FASTSWM run).

2) Change data using the normal EDIT processor.

3) Enter USE, FASTSWMM as before, but do not specify that a new file is to be written.

DESIGN CRITERIA FOR INTERACTIVE DIALOGUE

It has been our experience that the design time, including user transaction time and program turnaround time, are greatly reduced when the interactive approach to computing is taken [James and Robinson, 1978]. It has been found that concise, unambiguous man-machine dialogue promotes a reduction in user transaction time, minimizes the potential for error, increases user confidence and encourages investigation of different design situations. It was also found that transmitting only key output to the terminal enhances convergence to a final design.

More significant than computing cost is the cost of the designer's time. For example, for a batch program that requires one day's effort, depending on the designer's capability, a savings of about 85% of turnaround time can be achieved using an interactive package [James and Zachar, 1974]. The costs of that time difference may be quite significant to consultant and client alike, especially from remote small offices.

The importance of making the numerical model work for the user cannot be stressed too greatly. If the method of communicating with the computer is complicated and exacting, or the dialogue ambiguous, the positive aspects of computing will be nullified. There are three levels at which attention should be paid to user psychology - the functional level, the procedural level and the syntactical level [Robinson, 1977].

At the functional level, it must be made clear which functions are to be carried out by the designer and which functions by the computer. A brief introduction at the beginning of the program is important. In FASTSWMM the user is informed that he must supply data whenever three dots (...) appear. He is made aware that the computer will do all the prompting. The user can devote his time to the design problem.

The relative merits of batch and demand processing warrant discussion. The topic was converted briefly in an earlier report [Proctor et al, 1976]. In batch processing a run is submitted to the computer through a device such as a card-reader or keyboard terminal, and the job is then run and the output returned to the user either within an hour or so, or at some later time, with no communication between the user and the program from the time of submission to the receipt of results. Demand computing is initiated through a teletype-type terminal, but the program will halt at certain predetermined points to permit the user to make decisions before the run is continued. One further term should be noted, i.e. conversational computing. This is similar to demand computing, but specific prompts are written into the program in an effort to guide the user in making his decisions. Whether batch or demand computing should be used depends on whether or not it is important for the engineer to make decisions, independent of the computer, in the design function which he is attempting to carry out.

It is of course normally possible to access SWMM files via keyboard terminals using existing job control software, i.e. without use of FASTSWMM. This could permit an engineer to set up a major run in a remote batch mode and then, having completed a first run, modify certain input parameters via a keyboard terminal, retain the rest of the data unchanged, and initiate another run. Many users of SWMM use this method [Proctor and Redfern, 1976]. Use of FASTSWMM seems particularly attractive to neophyte users of SWMM. This paper presents the total urban design problem, in the context of SWMM, so that the reader may himself decide whether a program such as FASTSWMM would be useful to him in his circumstances. But there seems to be little doubt that the philosophy is useful in an instructional/training environment.

INSTRUCTIONAL VALUE OF FASTSWMM

Perhaps the most important attribute of conversational pre- and post-processors is that their use facilitates rapid assimilation of the SWMM program without learning effort being expended on variable names or input formats.

This makes it very easy to provide instructional courses on SWMM using FASTSWMM. Such a course has now being given at two locations in Sweden and in Toronto, each course lasting 2 to 3 days.

The main point of the course is to give the participants "hands-on" experience with a small part of the SWMM program. In other words the participants are expected to work from actual plans for a real urban area, discretize the areas themselves, prepare the input data, apply an actual recorded storm, and run the flow hydrograph part of the SWMM runoff block many times to gain a "feel" for the model.

Because of the complexity of the SWMM model, it is only possible in the time available to make a detailed study of hydrograph generation using the runoff block; participants are expected to have a good idea of the runoff block processes by the end of the course. It is expected that this experience will help the participants to attempt the other 4 or 5 blocks. Use of the FASTSWMM package has proven especially beneficial when the central processor unit is some distance away.

Completion of the course is deemed to include satisfactory attendance at the lectures and workshops, reasonable completion of the quizzes, and submission of a final report summarising achievements in the workshops.

The course workshop objectives are as follows:

Workshop 1: Select a problem area.
Study maps.
Mark off problem area divide.
Select subcatchments.
Measure areas and lengths.
Abstract data.
Select a design storm.

Workshop 2: Enter rough data by means of PRESWMM in the FASTSWMM package.
Edit data file.
Submit a SWMM job.

Workshop 3: Redesign the pipe network or
Select another level of discretization or
Select parameters for sensitivity tests.
Edit the input file.
Submit SWMM jobs.
Write final report.

Reading materials have been prepared and are assigned to the lectures as shown in Table 1. The contents of the lectures are also described in that table, and the reference materials are listed at the end of this paper.

CONCLUSIONS

Most users are faced with a number of realities:

(a) They cannot afford to develop and/or maintain an independent urban runoff model.

(b) They do not own a computer large enough to run a good urban runoff model.

(c) They have good telephone access to such computers and can afford a portable terminal.

(d) They are discouraged by the long learning-time required to become familiar with good urban runoff models.

(e) They would like to reduce the total design turnaround times associated with batch computing.

A program called FASTSWMM has been developed to cope with this environment. The main design principles were that:

(a) It should communicate directly with a standard SWMM.

(b) SWMM must be easily updated locally on receipt of updates from the original authors.

(c) SWMM should run without long delays because of excessive memory requirements.

(d) FASTSWMM should operate optionally in SI units but talk to FASTSWMM in the required FPS units.

(e) FASTSWMM should function on portable slow-speed terminals in a conversational manner.

(f) It should be easy to include other models such as STORM.

Thus no code has been inserted into SWMM except that required to make isolated statements transportable (about half-a-dozen statements).

The operations of FASTSWMM are:

1. CONVERSATIONAL INPUT
2. PROVIDE ROUGH DATA CHECKS
3. SI/FPS CONVERSION
4. ARCHIVE INPUT
5. CONNECT INPUT AND OBSERVED FLOWDATA FILES
6. RUN SWMM (& OTHER MODELS)

7. ACCESS OUTPUT
8. FPS/SI CONVERSION
9. PRINT SUMMARY OUTPUT
10. PROVIDE SIMPLE OUTPUT ANALYSIS
11. FULL REGULAR BATCH LISTINGS

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The writer spent a sabbatical year (July 1977-June 1978) in Sweden, the first half at the Institute for Water Resources Research (TVRL) at Lund Institute of Technology (LTH), and the second with a similar group at Lulea University. This work would not have been possible without the help of Gunnar Lindh, Lennart Jonsson and especially Jan Falk, at TVRL, and of Gunnar Peterson at the LTH Computer Centre. Help from Lars Bengtsson at Lulea is gratefully acknowledged.

Sabbatical leave from McMaster University is also duly acknowledged here, and the privilege of being in Sweden for a year.

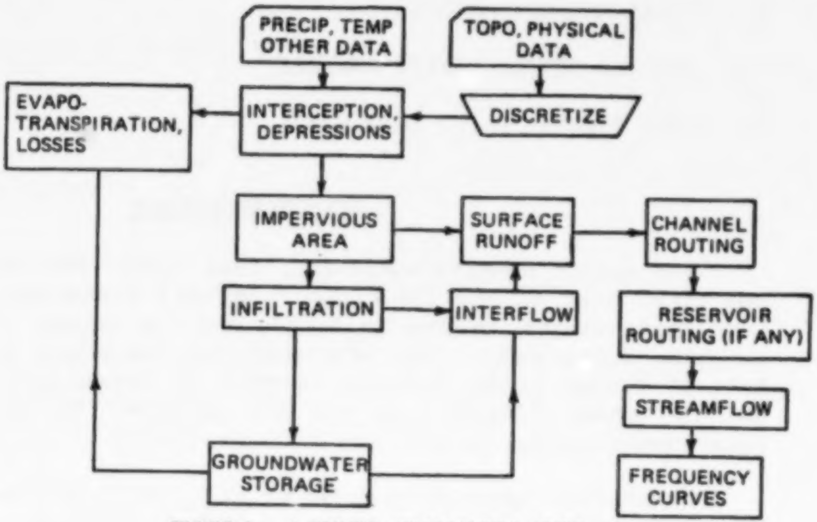
PRECIP, TEMP
OTHER DATA
TOPO, PHYSICAL
DATA
EVAPO-
TRANSPIRATION,
LOSSES
INTERCEPTION,
DEPRESSIONS
DISCRETIZE
IMPERVIOUS
AREA
SURFACE
RUNOFF
CHANNEL
ROUTING
INFILTRATION
INTERFLOW
RESERVOIR
ROUTING (IF ANY)
STREAMFLOW
GROUNDWATER
STORAGE
FREQUENCY
CURVES

FIGURE 1: A GENERAL HYDROLOGICAL MODEL
(ADAPTED FROM LINSLEY AND CRAWFORD)

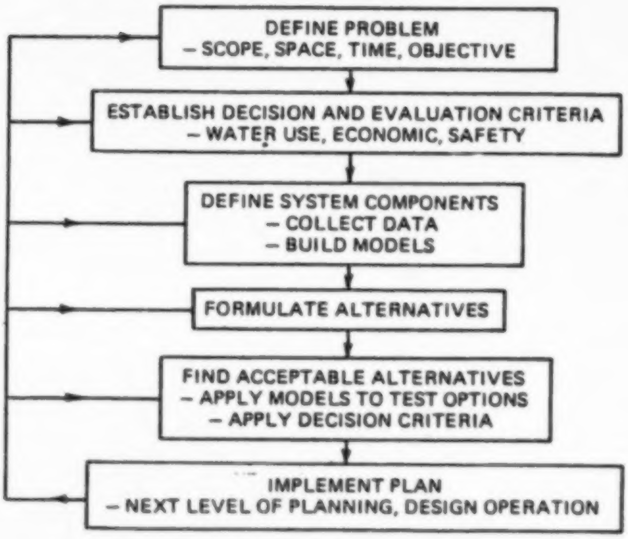
DEFINE PROBLEM
– SCOPE, SPACE, TIME, OBJECTIVE
ESTABLISH DECISION AND EVALUATION CRITERIA
– WATER USE, ECONOMIC, SAFETY
DEFINE SYSTEM COMPONENTS
– COLLECT DATA
– BUILD MODELS
FORMULATE ALTERNATIVES
FIND ACCEPTABLE ALTERNATIVES
– APPLY MODELS TO TEST OPTIONS
– APPLY DECISION CRITERIA
IMPLEMENT PLAN
– NEXT LEVEL OF PLANNING, DESIGN OPERATION

FIGURE 2: URBAN DRAINAGE DESIGN SCHEME
(ADAPTED FROM THE M.O.E. DRAINAGE MANUAL)

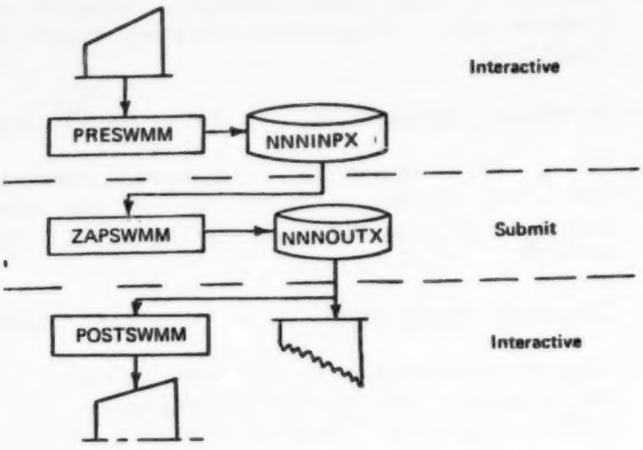
PRESWMM
NNNINPX
Interactive
ZAPSWMM
NNNOUTX
Submit
POSTSWMM
Interactive

FIGURE 3: FASTSWMM FILES

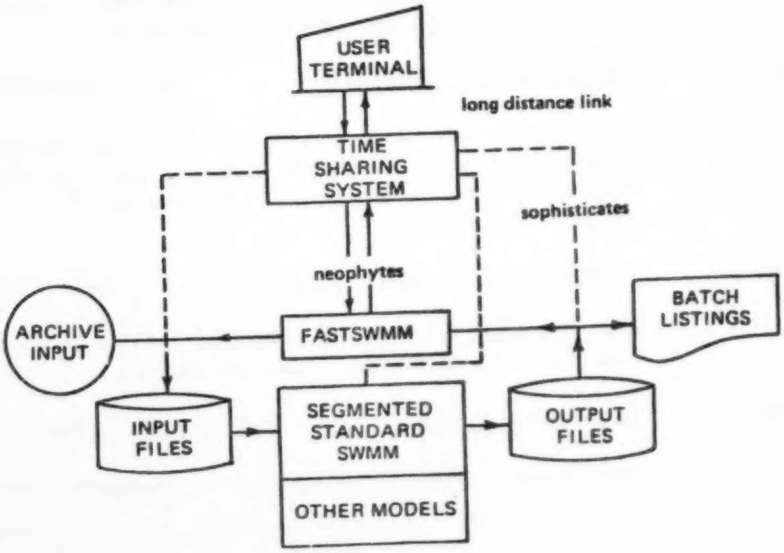
USER TERMINAL
long distance link
TIME SHARING SYSTEM
sophisticates
neophytes
ARCHIVE INPUT
FASTSWMM
BATCH LISTINGS
INPUT FILES
SEGMENTED STANDARD SWMM
OTHER MODELS
OUTPUT FILES

FIGURE 4: ACCESS TO FASTSWMM

TABLE 1

Lecture 1:	Introduction	Ref: Developing Models Vol. 1	Course outline reviewed. Various hydrological programs and their inter-relationship. Program development and program use. Comparison between batch and interactive computing.
Lecture 2:	Accessing the Computer	Ref: Developing Models Vol. 2	Using the terminals. General description of the operating system. Operation of the demand-processing system. Editing files at the terminal. Typical FASTSWMM session.
Lecture 3:	Introduction to SWMM	Ref: SWMM documentation	General arrangement of the SWMM package. Segmentation of the program. Sources of information. The EXEC, COMBINE, RUNOFF, TRANSPORT, STORAGE and RECEIVE blocks.
Lecture 4:	RUNOFF block flow hydrographs	Ref: SWMM documentation	Discretization of the catchments. Setting up the drainage schematization. Surface runoff hydrology. Infiltration and losses. Surface runoff algorithm. Routing through the drainage network.
Lecture 5:	FASTSWMM job submission	Ref: FASTSWMM Users Guide	The FASTSWMM concept. The input routine. Files associated with FASTSWMM jobs. Editing the input file. Repeated job submission. Accessing the complete batch output listing.
Lecture 6:	RUNOFF block capabilities	Ref: Water Resources	The Snowmelt algorithm. Quality aspects. Calibration of the flow hydrographs. Design storms.
Lecture 7:	Continuous SWMM	Ref: SWMM3 Interim documentation	Continuous SWMM simulation. Snowmelt modeling. The EXTRAN block.
Last Session:	Course Wrap-Up	Ref: All Course Materials	Review of whole course. Assessment of course. Suggestions for improvements. Suggestions for follow-up course. Suggestions for further work.

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